# SE-006 Design of Residential Foundations

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# CHAPTER 4

# Design of Foundations

## 4.1 General

A foundation supports and anchors the superstructure of a building and transfers all loads (including those from flood, wind, or seismic events) imposed on it directly to the ground. Foundations distribute the loads to the earth over an adequate area so that loads do not exceed the bearing capacity of the soil and so that lateral movement or settlement is minimized. In cold climates, the bottom of the foundation must be below the frost line to prevent freeze-thaw damage and frost heave of the footing.

A foundation in residential construction may consist of a footing, wall, slab, pier, pile, or a combination of these elements. This chapter addresses the following foundation types—

- Crawl space.
- Basement.
- Slab-on-grade with stem wall.
- Monolithic slab.
- Piles.
- Piers.
- Alternative methods.

As discussed in chapter 1, the most common residential foundation materials are cast-in-place concrete and concrete masonry (that is, concrete block). Preservative-treated wood, precast concrete, and other materials may also be used. The concrete slab-on-grade is a prevalent foundation type in the South and Southwest; basements are the most common type in the East and Midwest. Crawl spaces are common in the Northwest and Southeast. Pile foundations designed to function after being exposed to scour and erosion are commonly used in coastal flood zones to elevate structures above flood levels. Piles also are used in weak or expansive soils to reach a stable stratum and on steeply sloped sites. Figure 4.1 depicts different foundation types; a brief description follows.

A *crawl space* is a building foundation that uses a perimeter foundation wall to create an underfloor space that is not habitable; the interior crawl space elevation may or may not be below the exterior finish grade. In mapped flood plains, a crawl space that has the interior grade below the exterior grade on all sides is considered a basement. A *basement* typically is defined as a portion of a building that is partly or completely below the exterior grade and that may be used as habitable space, as storage space, or for parking. The primary difference between a basement and a crawl space is height (basements usually are taller). The floors of basements usually are finished, and the interiors frequently are finished. The wall height is sometimes determined by the depth of the footing required for frost protection.

A *slab-on-grade with an independent stem wall* is a concrete floor supported by the soil independently of the rest of the building. The stem wall supports the building loads and in turn is supported directly by the soil or a footing. A *monolithic* or *thickened-edge slab* is a ground-supported slab-on-grade with an integral footing (that is, a thickened edge); it is normally used in warmer regions that have little or no frost depth but is also used in colder climates when adequate frost protection is provided (see section 4.7).

When necessary, *piles* are used to transmit the load to a deeper soil stratum with a higher bearing capacity to prevent failure from undercutting of the foundation by scour from flood water flow at high velocities and to elevate the building above required flood elevations. Piles also are used to isolate the structure from expansive soil movements.

*Pier and beam foundations* can provide an economical alternative to crawl space perimeter wall construction. A common practice is to use a brick curtain wall between piers for appearance and bracing.

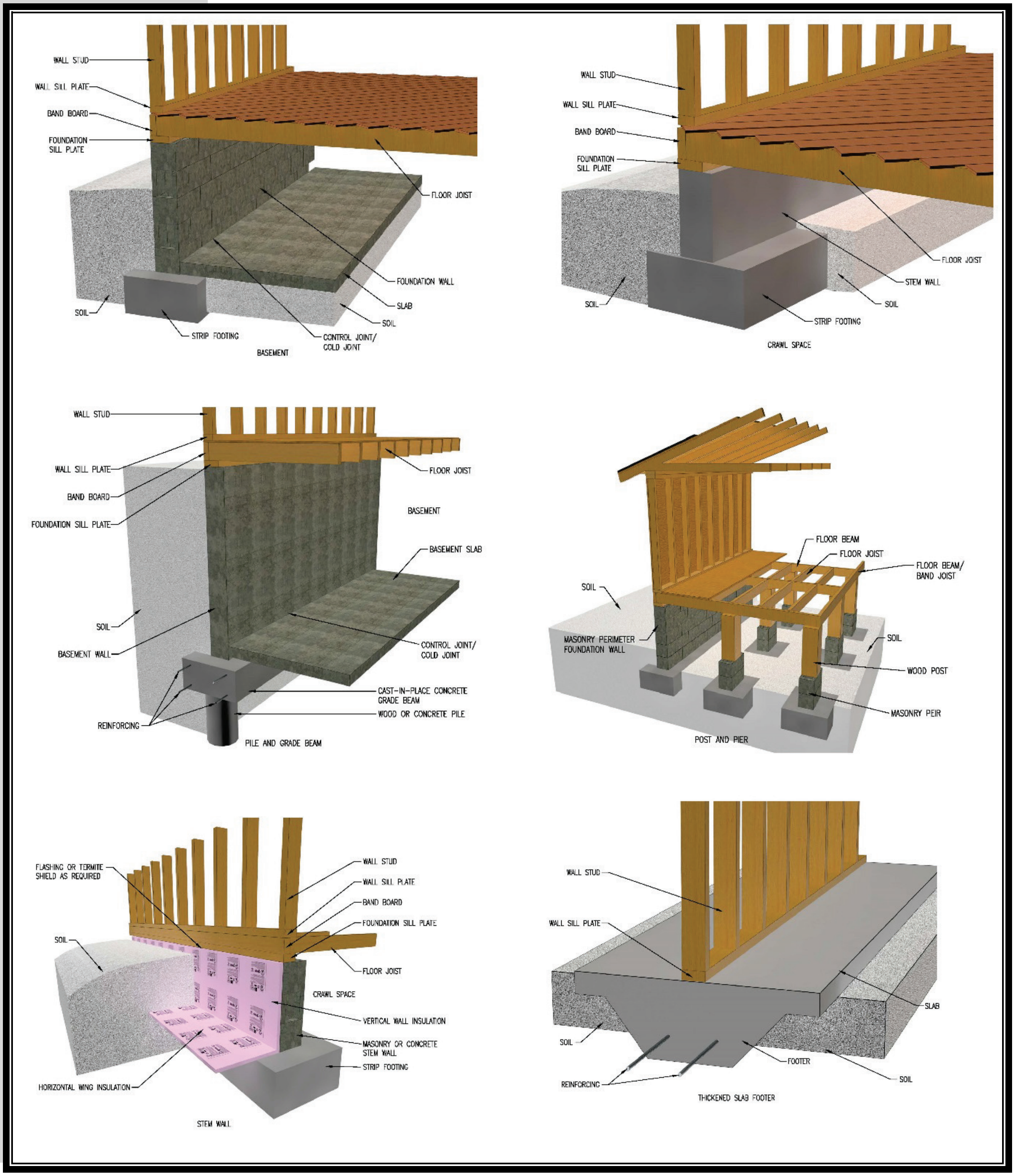
The design procedures and information in this chapter cover the following topics.

- Foundation materials and properties.
- Soil-bearing capacity and footing size.
- Concrete or gravel footings.
- Concrete and masonry foundation walls.
- Preservative-treated wood walls.
- Insulating concrete foundations.
- Concrete slabs on grade.
- Pile foundations.
- Frost protection.

Concrete design procedures generally follow the strength design method contained in the American Concrete Institute's ACI 318 (ACI, 2011) although certain aspects of the procedures may be considered conservative relative to conventional residential foundation applications. For this reason, this guide provides supplemental design guidance when practical and technically justified. ACI 332 (ACI, 2010), which contains design provisions and guidance specific to residential construction, is referenced in the International Residential Code (IRC) as an alternative to the conventional foundation requirements of the code or the design procedures of ACI 318. Masonry design procedures follow the allowable stress design (ASD) method of The Masonry Society's TMS 402 (TMS, 2011). Wood design procedures are used to design the connections between the foundation system and the structure above and follow the ASD method for wood construction (see chapter 7 for connection design information). In addition, the designer is referred to the applicable design standards for symbol definitions and additional guidance because the intent of this chapter is to provide supplemental instruction in the efficient design of residential foundations.

To maintain consistency, this guide uses the load and resistance factor design (LRFD) load combinations that were used in chapter 3, which are also those specified in the American Society of Civil Engineers' ASCE 7. There may be some minor variations in those required in ACI 318 for strength design of concrete. The purpose of this guide is to provide designs that are at least consistent with current residential building code and construction practice. With respect to the design of concrete in residential foundations, the guide seeks to provide reasonable safety margins that meet or exceed the minimums required for other, more crucial requirements of a home—namely, the safety of lives. The designer is responsible for ensuring that the design meets the building code requirements and will be approved by the building official.

## *FIGURE 4.1 Types of Foundations*



# 4.2 Material Properties

A residential designer using concrete and masonry materials must have a basic understanding of such materials, including variations in the materials' composition and structural properties. In addition, a designer must take into consideration soils, which are also considered a foundation material (Section 4.3 provides information on soil bearing). A brief discussion of the properties of concrete and masonry follows.

## 4.2.1 Concrete

The concrete compressive strength ($f_c'$) used in residential construction is typically either 2,500 or 3,000 pounds per square inch (psi), although other values may be specified. For example, 4,000 psi concrete may be used for improved weathering resistance in particularly severe climates or unusual applications. The concrete compressive strength may be verified in accordance with the American Society for Testing and Materials' ASTM C39 (ASTM, 2012c). Given that the rate of increase in concrete strength diminishes with time, the specified compressive strength usually is associated with the strength attained after 28 days of curing time, when the concrete attains about 85 percent of its fully cured compressive strength.

Concrete is a mixture of cement, water, and sand, gravel, crushed rock, or other aggregates. Sometimes one or more admixtures are added to change certain characteristics of the concrete, such as workability, durability, and time of hardening. The proportions of the components determine the concrete mix's compressive strength and durability.

***Type***

Portland cement is classified into several types, in accordance with ASTM C150 (ASTM, 2012b). Residential foundation walls typically are constructed with *Type I* cement, which is a general-purpose Portland cement used for the vast majority of construction projects. Other types of cement are appropriate in accommodating conditions related to heat of hydration in massive pours and sulfate resistance. In some regions, sulfates in soils have caused durability problems with concrete. The designer should check into local conditions and practices.

***Weight***

The weight of concrete varies depending on the type of aggregates used in the concrete mix. Concrete typically is classified as lightweight or normal weight. The density of unreinforced normal weight concrete ranges between 144 and 156 pounds per cubic foot (pcf) and typically is assumed to be 150 pcf. Residential foundations usually are constructed with normal weight concrete.

### *Slump*

Slump is the measure of concrete consistency; the higher the slump, the wetter the concrete and the easier it flows. Slump is measured in accordance with ASTM C143 (ASTM, 2012d) by inverting a standard 12-inch-high metal cone, filling it with concrete, and then removing the cone; the amount the concrete that settles in units of inches is the slump. Most foundations, slabs, and walls consolidated by hand methods have a slump between 4 and 6 inches. One problem associated with a high-slump concrete is segregation of the aggregate, which leads to cracking and scaling. Therefore, a slump of greater than 6 inches should be avoided. Adding water lowers the strength while improving workability, so the total amount of water in the concrete should be carefully monitored and controlled. Admixtures used during extremely cold or hot weather placement (or for other reasons) may change the slump.

### *Weather Resistance*

Concrete is largely weather resistant. When concrete may be subjected to freezing and thawing during construction, however, or when concrete is located in regions prone to extended periods of freezing, additional measures must be taken. Those requirements can be found in the IRC (ICC, 2012), and include air entrainment and increased minimum compressive strength requirements.

### *Admixtures*

Admixtures are materials added to the concrete mix to improve workability and durability and to retard or accelerate curing. Some of the most common admixtures are described below.

- *Water reducers* improve the workability of concrete without reducing its strength.
- *Retarders* are used in hot weather to allow more time for placing and finishing concrete. Retarders may also reduce the early strength of concrete.
- *Accelerators* reduce the setting time, allowing less time for placing and finishing concrete. Accelerators may also increase the early strength of concrete.
- *Air entrainers* are used for concrete that will be exposed to freeze-thaw conditions and deicing salts. Less water is needed and segregation of aggregate is reduced when air entrainers are added.

### *Reinforcement*

Concrete has high compressive strength but low tensile strength; therefore, reinforcing steel often is embedded in the concrete to provide additional tensile strength and ductility. In the rare event that the capacity is exceeded, the reinforcing steel begins to yield, thereby preventing an abrupt failure that may otherwise occur with plain, unreinforced concrete. For this reason, a larger safety margin is used in the design of plain concrete construction than in reinforced concrete construction.

Steel reinforcement is available in grade 40 or grade 60; the grade number refers to the minimum tensile yield strength ($f_y$) of the steel (i.e., grade 40 is a minimum 40 thousand pounds per square inch [ksi] steel and grade 60 is a minimum 60 ksi steel). Either grade may be used for residential construction; however, most steel reinforcement in the U.S. market today is grade 60. The concrete mix, or slump, must be adjusted by adding the appropriate amount of water to allow the concrete to flow easily around the reinforcement bars, particularly when the bars are closely spaced or are crowded at points of overlap. Close rebar spacing rarely is required in residential construction, however, and should be avoided in design if at all possible.

The most common steel reinforcement or rebar sizes in residential construction are No. 3, No. 4, and No. 5, which correspond to diameters of 3/8 inch, 1/2 inch, and 5/8 inch, respectively. The bar designations indicate the bar size in 1/8-inch increments. These three sizes of rebar are easily manipulated at the jobsite by using manual bending and cutting devices. Table 4.1 shows useful relationships between the rebar number, diameter, and cross-sectional area for reinforced concrete and masonry design.

Fiber reinforcement is being used in some concrete slab installation. The fiber could be steel, natural, or synthetic. It helps (1) improve resistance to freeze-thaw, (2) increase resistance to some spalling of the surface, (3) control cracking, and (4) improve the concrete's shatter resistance. Fibers generally do not increase the structural strength of the concrete slab and do not replace normal reinforcing bars used for tensile strength.

**TABLE 4.1 Rebar Size, Diameter, and Cross-Sectional Areas**

| Size | Diameter (inches) | Area (square inches) |
|---|---|---|
| No. 3 | 3/8 | 0.11 |
| No. 4 | 1/2 | 0.20 |
| No. 5 | 5/8 | 0.31 |
| No. 6 | 3/4 | 0.44 |
| No. 7 | 7/8 | 0.60 |
| No. 8 | 1 | 0.79 |

## 4.2.2 Concrete Masonry Units

Concrete masonry units (CMUs), commonly referred to as concrete blocks, are composed of Portland cement, aggregate, and water. In some situations, CMUs may also include admixtures. Low-slump concrete is molded and cured to produce strong blocks or units. Residential foundation walls typically are constructed with units 7 5/8 inches (nominal 8 inches) high by 15 5/8 inches (nominal 16 inches) long, providing a 3/8-inch allowance for the width of mortar joints. Nominal 8- and 12-inch-thick CMUs are readily available for use in residential construction.

***Type***

ASTM C90 (ASTM, 2013) requires that the minimum average design strength ($f'_m$) of standard CMUs be 1,900 psi, with no individual unit having a compressive strength of less than 1,700 psi. Higher strengths also may be specified if required by design. The ASTM classification includes two types. Type II is a non-moisture-controlled unit and is the type typically used for residential foundation walls.

***Weight***

CMUs are available with different densities by altering the type(s) of aggregate used in their manufacture. CMUs typically are referred to as lightweight, medium weight, or normal weight, with respective unit weights or densities of less than 105 pcf, between 105 and 125 pcf, and more than 125 pcf. Residential foundation walls typically are constructed with low- to medium-weight units because of the low compressive strength required. Lower density units are generally more porous, however, and must be properly protected to resist moisture intrusion. A common practice in residential basement foundation wall construction is to provide a cement-based parge coating and a brush- or spray-applied bituminous coating on the belowground portions of the wall. Section R406 in the IRC provides the prescriptive requirements for parging and damp-proofing or waterproofing foundation walls that retain earth and enclose interior spaces. The parge coating is not required for concrete foundation wall construction.

***Hollow or Solid***

CMUs are classified as hollow or solid in accordance with ASTM C90. The net concrete cross-sectional area of most CMUs ranges from 50 to 70 percent, depending on unit width, face-shell and web thicknesses, and core configuration. *Hollow* units are defined as those in which the net concrete cross-sectional area is less than 75 percent of the gross cross-sectional area. *Solid* units are not necessarily solid but are defined as those in which the net concrete cross-sectional area is 75 percent of the gross cross-sectional area or greater.

### *Mortar*

Masonry mortar is used to join CMUs into a structural wall; it also retards air and moisture infiltration. The most common way to lay block is in a running bond pattern, in which the vertical head joints between blocks are offset by half the block length from one course to the next. Mortar is composed of water, cement, lime, and clean, well-graded sand, and water and is typically classified into *types M, S, N, O,* and *K,* in accordance with ASTM C270 (ASTM, 2012a). Residential foundation walls typically are constructed with *type M* or *type S* mortar, both of which are generally recommended for load-bearing interior and exterior walls, including above- and below-grade applications.

### *Grout*

Grout is a slurry consisting of cementitious material, aggregate, and water. When needed, grout commonly is placed in the hollow cores of CMUs to provide a wall with added strength. In reinforced load-bearing masonry wall construction, grout is usually placed only in those hollow cores containing steel reinforcement. The grout bonds the masonry units and steel so that they act as a composite unit to resist imposed loads. Grout may also be used in unreinforced concrete masonry walls for added strength. The IRC requires grouted cells at foundation sill and sole plate anchor bolt locations, regardless of whether the masonry wall is otherwise reinforced.

# 4.3 Soil-Bearing Capacity and Footing Size

Soil-bearing investigations rarely are required for residential construction except when a history of local problems provides evidence of known risks (for example, organic deposits, landfills, expansive soils, and seismic risk). Soil-bearing tests on stronger-than-average soils can, however, justify using smaller footings or eliminating footings entirely if the foundation wall provides sufficient bearing surface. Table 4.2 provides a conservative relationship between soil type and load-bearing value. A similar table is published in the building codes (table R401.4.1 in the IRC). These presumptive soil-bearing values, however, should be used only when the building codes do not require geotechnical investigation reports (section R401.4, IRC).

## *TABLE 4.2 Presumptive Soil-Bearing Values by Soil Description*

| Presumptive Load-Bearing Value (psf) | Soil Description |
|---|---|
| 1,500 | Clay, sandy clay, silty clay, clayey silt, silt, and sandy silt |
| 2,000 | Sand, silty sand, clayey sand, silty gravel, and clayey gravel |
| 3,000 | Gravel and sandy gravel |
| 4,000 | Sedimentary and foliated rock |
| 12,000 | Crystalline bedrock |

*psf = pounds per square foot.*

*Source: ICC (2012).*

When a soil-bearing investigation is desired to determine more accurate and economical footing requirements, the designer commonly turns to ASTM D1586, *Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils* (ASTM, 2011). This test relies on a 2-inch-diameter device driven into the ground with a 140-pound hammer dropped from a distance of 30 inches. The number of hammer drops or blows needed to create a 1-foot penetration (blow count) yields values that can be roughly correlated to soil-bearing values, as shown in Table 4.3. The instrumentation and cost of conducting the SPT usually are not warranted for typical residential applications. Nonetheless, the SPT method provides information on deeper soil strata and thus can offer valuable guidance for foundation design and building location, particularly when subsurface conditions threaten to be problematic. The values in Table 4.3 are associated with the blow count from the SPT method. Many engineers can provide reasonable estimates of soil bearing by using smaller penetrometers at lower cost, although such devices and methods may require an independent calibration to determine presumptive soil-bearing values and may not be able to detect deep subsurface problems. Calibrations may be provided by the manufacturer or, alternatively, developed by the engineer.

In addition to ASTM D1586, the Dynamic Cone Penetrometer (DCP) test (Burnham and Johnson, 1993), has gained widespread use as a more economical alternative with equivalent accuracy. In this handheld test, a metal cone is driven into the ground by repeatedly striking it with a 17.6-pound (8-kilogram) weight, dropped from a distance of 2.26 feet (575 millimeters). Penetration of the cone is measured after each blow; the blow count per 1 3/4-inch penetration is approximately equivalent to the SPT blow count provided in table 4.3.

The designer should exercise judgment when selecting the final design value and be prepared to make adjustments (increases or decreases) in interpreting and applying the results to a specific design. The values in tables 4.2 and 4.3 generally are associated with a safety factor of 3 (Naval Facilities Engineering Command, 1986) and are considered appropriate for noncontinuous or independent spread footings supporting columns or piers (that is, point loads). Use of a safety factor could be considered for smaller structures with continuous spread footings, such as houses, or structures for which ultimate (LRFD) values are used for design loads. The presumptive values in Table 4.3—or as modified as described previously—are intended to be used with the ASD load combinations in chapter 3. If LRFD (strength) design load combinations are used, then the presumptive values should be additionally adjusted (that is, divided by the maximum load factor in the load combination considered, usually a factor of 1.6 for live or snow loads).

***Table 4.3*** ***Presumptive Soil-Bearing Values (psf) Based on Standard Penetrometer Blow Count***

| In Situ Consistency, N[1] | | Loose[2] (5 to 10 blows per foot) | Firm (10 to 25 blows per foot) | Compact (25 to 50 blows per foot) |
|---|---|---|---|---|
| Noncohesive Soils | Gravel | 4,000 (10) | 8,000 (25) | 11,000 (50) |
| | Sand | 2,500 (6) | 5,000 (20) | 6,000 (35) |
| | Fine sand | 1,000 (5) | 3,000 (12) | 5,000 (30) |
| | Silt | 500 (5) | 2,000 (15) | 4,000 (35) |
| **In Situ Consistency, N[1]:** | | **Soft[3] (3 to 5 blows per foot)** | **Medium (about 10 blows per foot)** | **Stiff (more than 20 blows per foot)** |
| Cohesive Soils | Clay, sand, gravel mixtures | 2,000 (3) | 5,000 (10) | 8,000 (20) |
| | Sandy or silty clay | 1,000 (4) | 3,000 (8) | 6,000 (20) |
| | Clay | 500 (5) | 2,000 (10) | 4,000 (25) |

*Source: Naval Facilities Engineering Command, 1986.*

*psf = pounds per square foot.*

[1]N denotes the standard penetrometer blow count in blows per foot, in accordance with ASTM D1586; shown in parentheses.
[2]Compaction should be considered in these conditions, particularly when the blow count is five blows per foot or less.
[3]Pile and grade beam foundations should be considered in these conditions, particularly when the blow count is five blows per foot or less.

The required width or area of a spread footing is determined by dividing the building load on the footing by the soil-bearing capacity from table 4.2 or table 4.3, as shown below. Building design loads, including dead and live loads, should be determined in accordance with chapter 3 by using ASD load combinations.

$$Area_{independent\ spread\ footing} = \frac{Load\ in\ lbs}{Soil - bearing\ capacity\ in\ psf}$$

$$Width_{continuous\ footing} = \frac{Load,\ lb\ per\ linear\ foot\ (plf)}{Soil - bearing\ capacity\ in\ psf}$$

# 4.4 Footings

The objectives of footing design are—

- To provide a level surface for construction of the foundation wall.
- To provide adequate transfer and distribution of building loads to the underlying soil.
- To provide adequate strength, in addition to the foundation wall, to prevent differential settlement of the building in weak or uncertain soil conditions by bridging those poor soil conditions.
- To place the building foundation at a sufficient depth to avoid frost heave or thaw weakening in frost-susceptible soils and to avoid organic surface soil layers.
- To provide adequate anchorage or mass (when needed in addition to the foundation wall) to resist potential uplift, sliding, and overturning forces resulting from high winds or severe seismic events.

This section presents design methods for concrete and gravel footings. The designer must first establish the required footing width in accordance with section 4.3. Further, if soil conditions are stable or the foundation wall can adequately resist potential differential settlement, the footing may be completely eliminated.

By far, the most common footing in residential construction is a continuous concrete spread footing; however, concrete and gravel footings are both recognized in prescriptive footing size tables in residential building codes for most typical conditions (ICC, 2012). In contrast, special conditions give rise to engineering concerns that must be addressed to ensure the adequacy of any foundation design. Special conditions include—

- Steeply sloped sites requiring a stepped footing.
- High wind conditions.
- Inland or coastal flooding conditions.
- High-hazard seismic conditions.
- Poor soil conditions.

### 4.4.1 Simple Gravel and Concrete Footing Design

Building codes for residential construction contain tables that prescribe minimum footing widths for plain concrete footings (ICC, 2012). Alternatively, footing widths may be determined in accordance with section 4.3, based on a site's particular loading condition and presumptive soil-bearing capacity. The following are general rules of thumb for determining the thickness of plain concrete footings for residential structures, once the required bearing width has been calculated.

- The minimum footing thickness should not be less than the distance the footing extends outward from the edge of the foundation wall or 6 inches, whichever is greater.
- The footing width should project a minimum of 2 inches from both faces of the wall (to allow for a minimum construction tolerance) but not greater than the footing thickness.

These rules of thumb generally result in a footing design that differs somewhat from the plain concrete design provisions of chapter 22 of ACI 318. Footing widths generally follow the width increments of standard excavation equipment (in other words, a backhoe bucket size of 12, 16, or 24 inches). Although longitudinal steel reinforcement is not always required for residential-scale structures in typical soil conditions, the designer should consider adding some (two No. 4 or No. 5 bars is common) to avoid possible footing cracking where soil consolidation or a loss of soil-bearing capacity can occur. For situations in which the rules of thumb or prescriptive code tables do not apply or in which a more economical solution is possible, a more detailed footing analysis may be considered (see section 4.4.2). Example 4.1 in section 4.9 illustrates a plain concrete footing design in accordance with the simple method described herein.

Much like a concrete footing, a gravel footing may be used to distribute foundation loads to a sufficient soil-bearing surface area. A gravel footing provides a continuous path for water or moisture and thus must be drained in accordance with the foundation drainage provisions of the IRC. Gravel footings are constructed of crushed stone or gravel that is consolidated by tamping or vibrating. Pea gravel, which is naturally consolidated, does not require compaction and can be screeded to a smooth, level surface, much like concrete. Although typically associated with pressure-treated wood foundations (refer to section 4.5.3), a gravel footing can support cast-in-place or precast concrete foundation walls.

The size of a gravel footing usually is based on a 30- to 45-degree angle of repose for distributing loads; therefore, as with plain concrete footings, the required depth and width of the gravel footing depends on the width of the foundation wall, the foundation load, and soil-bearing values. Following a rule of thumb similar to that for a concrete footing, the gravel footing thickness should be no less than 1.5 times its extension beyond the edge of the foundation wall or, in the case of a pressure-treated wood foundation, the mud sill. Just as with a concrete footing, the thickness of a gravel footing may be considered in meeting the required frost depth. In soils that are not naturally well drained, provision should be made to adequately drain a gravel footing.

## 4.4.2 Concrete Footing Design

For many residential footing designs, prescriptive and conventional residential footing requirements found in residential building codes and construction guides are adequate, if not conservative. Concrete design for residential construction is covered in ACI 332 *Residential Code Requirements for Structural Concrete* (ACI, 2010). To improve performance and economy or to address peculiar conditions, however, a footing may need to be specially designed. Many floor plans in today's residential buildings are partially open and frequently create nonuniform loading conditions on load-bearing walls and footings. These nonuniform load conditions must be considered in the design of footings, and reliance on strictly prescriptive methods of design is not always a sound design decision.

A footing is designed to resist the upward-acting pressure created by the soil beneath the footing; that pressure tends to make the footing bend upward at its edges. According to ACI 318, the three modes of failure considered in reinforced concrete footing design are one-way shear, two-way shear, and flexure (see figure 4.2). Bearing (crushing) is also a possible failure mode but is rarely applicable to residential loading conditions. To simplify calculations for the three failure modes, the following discussion explains the relation of the failure modes to the design of plain and reinforced concrete footings (Refer to ACI 318 for additional commentary and guidance). The design equations used later in this section are based on ACI 318 and principles of engineering mechanics, as described herein. Moreover, the approach is based on the assumption of uniform soil-bearing pressure on the bottom of the footing; therefore, walls and columns should be supported as close as possible to the center of the footings.

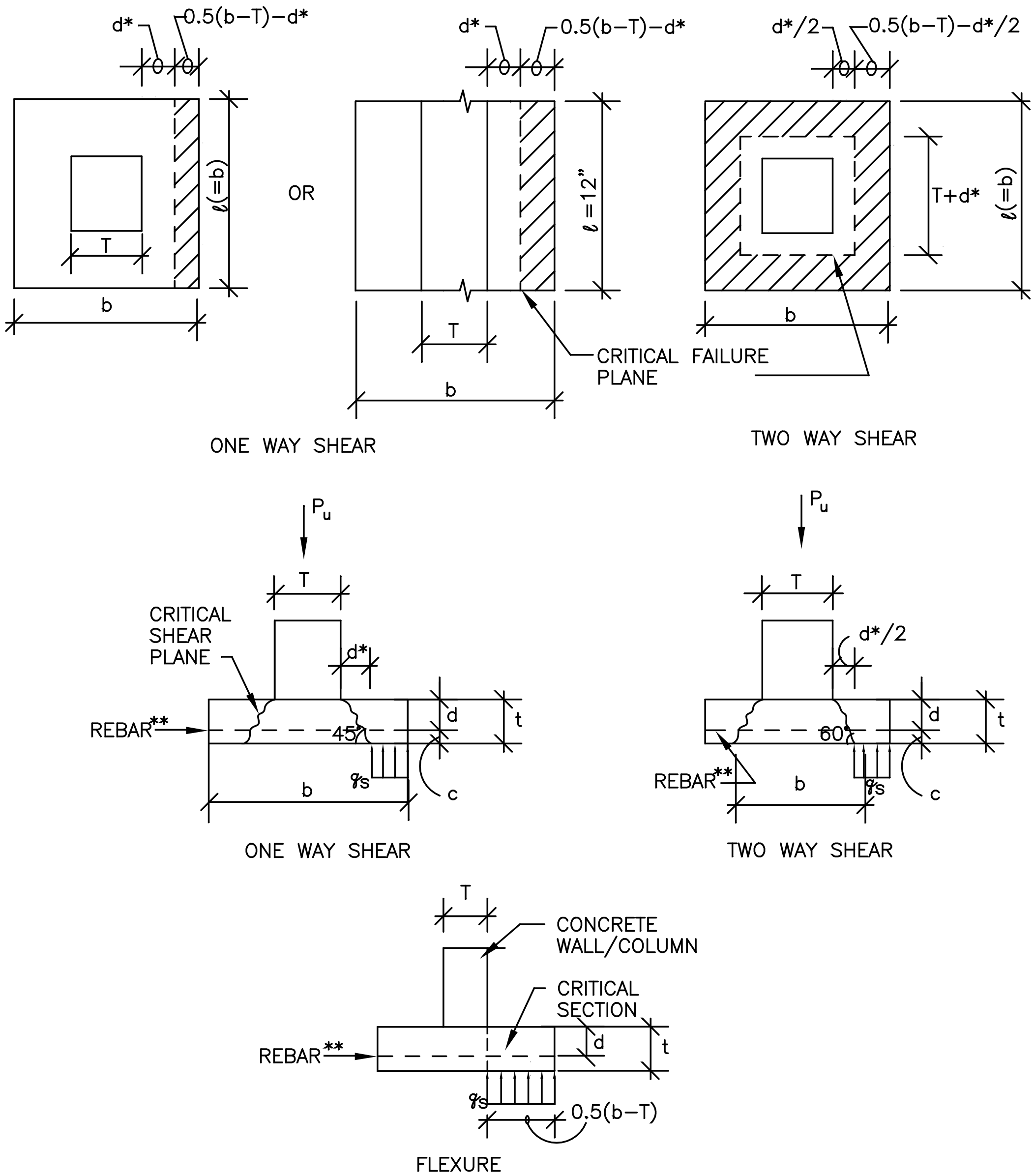
***One-Way (Beam) Shear***

When a footing fails due to *one-way (beam) shear*, the failure occurs at an angle approximately 45 degrees to the wall, as shown in figure 4.2. For plain concrete footings, the soil-bearing pressure has a negligible effect on the diagonal shear tension for distance $t$ from the wall edge toward the footing edge; for reinforced concrete footings, the distance used is $d$, which equals the depth to the footing rebar (see figure 4.2). As a result, one-way shear is checked by assuming that beam action occurs at a critical failure plane extending across the footing width, as shown in figure 4.2. One-way shear must be considered in similar fashion in both continuous wall and rectangular footings; however, for ease of calculation, continuous wall footing design typically is based on one lineal foot of wall or footing.

### *Two-Way (Punching) Shear*

When a footing fails by *two-way (punching) shear*, the failure occurs at an angle approximately 30 degrees to the column or pier, as shown in figure 4.2. Punching shear rarely is a concern in the design of continuous wall footings; thus, punching shear is usually checked only in the case of rectangular or circular footings with a heavily loaded pier or column that creates a large concentrated load on a relatively small area of the footing. For plain concrete footings, the soil-bearing pressure has a negligible effect on the diagonal shear tension at distance $t/2$ from the face of a column toward the footing edges; for reinforced concrete footings, the distance from the face of the column is $d/2$ (see figure 4.2). The shear force, therefore, consists of the net upward-acting pressure on the area of the footing outside the punched-out area (hatched area in figure 4.2). For square, circular, or rectangular footings, shear is checked at the critical section that extends in a plane around a concrete, masonry, wood, or steel column or pier that forms the perimeter $b_o$ of the area previously described.

## *FIGURE 4.2 Critical Failure Planes in Continuous or Square Concrete Spread Footings*




NOTES: * SUBSTITUTE t FOR d AS REQUIRED FOR PLAIN CONCRETE FOOTING DESIGN

** REBAR IS REQUIRED ONLY IN REINFORCED CONCRETE FOOTING DESIGN AND IS SHOWN HERE FOR THAT PURPOSE ONLY. IN REINFORCED SQUARE FOOTINGS, THE REBAR MUST BE PLACED IN TWO DIRECTIONS.

*Flexure (Bending)*

The maximum moment in a footing deformed by the upward-acting soil pressures would logically occur in the middle of the footing; however, the rigidity of the wall or column above resists some of the upward-acting forces and affects the location of maximum moment. As a result, the critical flexure plane for footings supporting a rigid wall or column is assumed to be located at the face of the wall or column. Flexure in a concrete footing is checked by computing the moment created by the soil-bearing forces acting over the cantilevered area of the footing that extends from the critical flexure plane to the edge of the footing (hatched area in figure 4.2). The approach for masonry walls in ACI 318 differs slightly in that the failure plane is assumed to be located one-fourth of the way under a masonry wall or column, creating a slightly longer cantilever. For the purpose of this guide, the difference is considered unnecessary.

*Bearing Strength*

Conditions in which concrete bearing or compressive strength is a concern are uncommon in typical residential construction; therefore, a design check usually can be dismissed as "OK by inspection." In rare and peculiar instances in which bearing compressive forces on the concrete are extreme and approach or exceed the specified concrete compressive strength, the designer should consult ACI 318 for appropriate guidance.

## 4.4.2.1 Plain Concrete Footing Design

In this section, the design of plain concrete footings is presented by using the concepts related to shear and bending covered in the previous section (refer to example 4.1 in section 4.9 for a design example of a plain concrete footing).

*Shear*

In the equations that follow for one- and two-way shear, the dimensions are in accordance with figure 4.2; units of inches should be used. ACI 318 requires that the overall thickness (t) be taken as 2 inches less than the actual thickness to compensate for uneven trench conditions. The following equations are specifically tailored for footings supporting walls or square columns because such footings are common in residential construction. The equations may be generalized for use with other conditions (for example, rectangular footings and rectangular columns, round footings) by following the same principles. In addition, the terms $4/3\sqrt{f'_c}$ and $4\sqrt{f'_c}$ are in units of psi and represent lower bound estimates of the ultimate shear stress capacity of unreinforced concrete.

ACI 318•22.5,22.7

One-Way (Beam) Shear

| | |
|---|---|
| $\phi V_c \geq V_u$ | basic design check for shear |
| $V_u = (q_s)(0.5(b - T) - t)\ell$ | factored shear load (lb) |
| $q_s = \dfrac{P_u}{b\ell}$ | uniform soil bearing pressure (psi) due to factored foundation load $P_u$ (lb) |
| $\phi V_c = \phi \dfrac{4}{3}\sqrt{f'_c}\,\ell t$ | factored shear capacity (lb) |
| $\phi = 0.65$ | resistance factor |

Two-Way (Punching) Shear

| | |
|---|---|
| $\phi V_c \geq V_u$ | basic design check for shear |
| $V_u = (q_s)\left(b\ell - (T + t)^2\right)$ | shear load (lb) due to factored load $P_u$ (lb) |
| $q_s = \dfrac{P_u}{b\ell}$ | uniform soil bearing pressure (psi) due to factored foundation load $P_u$ (lb) |
| $\phi V_c = \phi 4\sqrt{f'_c}\, b_o t$ | factored shear capacity (lb) |
| $b_o = 4(T + t)$ | perimeter of critical failure plane around a square column or pier |
| $\phi = 0.65$ | resistance factor |

### *Flexure*

For a plain concrete footing, flexure (bending) is checked by using the equations that follow for footings that support walls or square columns (see figure 4.2). The dimensions in the equations are in accordance with figure 4.2 and use units of inches. The term $5\sqrt{f'_c}$ is in psi and represents a lower bound estimate of the ultimate tensile (rupture) stress of unreinforced concrete in bending.

ACI 318•22.5,22.7

| | |
|---|---|
| $\phi M_n \geq M_u$ | basic design check for bending |
| $M_u = \frac{1}{8} q_s \ell (b - T)^2$ | factored moment (in-lb) due to soil pressure $q_s$ (psi) acting on cantilevered portion of footing |
| $q_s = \frac{P_u}{bl}$ | uniform soil bearing pressure (psi) due to factored load $P_u$ (lb) |
| $\phi M_n = \phi 5 \sqrt{f'_c} S$ | factored moment capacity (in-lb) for plain concrete |
| $S = \frac{1}{6} \ell t^2$ | section modulus (in$^3$) for footing |
| $\phi = 0.65$ | resistance factor for plain concrete in bending |

## 4.4.2.2 Reinforced Concrete Footing Design

For situations in residential construction in which a plain concrete footing may not be practical or in which reducing the footing thickness is more economical, steel reinforcement should be considered. A reinforced concrete footing is designed similar to a plain concrete footing; however, the concrete depth *d* to the reinforcing bar is used to check shear instead of the entire footing thickness *t*. The depth of the rebar is equal to the thickness of the footing minus the diameter of the rebar $d_b$ and the concrete cover *c*. In addition, the moment capacity is determined differently due to the presence of the reinforcement, which resists the tension stresses induced by the bending moment. Finally, a higher resistance factor reflects the more consistent bending strength of reinforced concrete compared to unreinforced concrete.

As specified by ACI 318, a minimum of 3 inches of concrete cover over steel reinforcement is required when concrete is in contact with soil. In addition, ACI 318 does not permit a depth *d* less than 6 inches for reinforced footings supported by soil. The designer may relax these limits, provided that the strength analysis demonstrates adequate capacity; however, a reinforced footing thickness of significantly less than 6 inches may be considered impractical, even though it may calculate acceptably. One exception may be found where a nominal 4-inch-thick slab is reinforced to serve as an integral footing for an interior load-bearing wall (which is not intended to transmit uplift forces from a shear wall overturning restraint anchorage in high-hazard wind or seismic regions). Further, the concrete cover should not be less than 2 inches for residential applications, although this recommendation may be somewhat conservative for interior footings that are generally less exposed to ground moisture and other corrosive agents. Example 4.2 of section 4.9 illustrates reinforced concrete footing design. The placement of steel to comply with concrete cover requirements may also significantly reduce the depth of steel, thus reducing flexural capacity of the concrete element; the designer must consider this reduced depth of steel.

### *Shear*

In the following equations for one- and two-way shear, the dimensions are in accordance with figure 4.2; units of inches should be used. Shear reinforcement (that is, stirrups) is usually considered impractical for residential footing construction; therefore, the concrete is designed to withstand the shear stress, as expressed in the equations. The equations are specifically tailored for footings supporting walls or square columns because such footings are common in residential construction. The equations may be generalized for use with other conditions (rectangular footings and rectangular columns, round footings, and so on) by following the same principles. In addition, the terms $2\sqrt{f'_c}$ and $4\sqrt{f'_c}$ are in units of psi and represent lower bound estimates of the ultimate shear stress capacity of reinforced concrete.

ACI 318•11.12,15.5

One-Way (Beam) Shear

| | |
|---|---|
| $\phi V_c \geq V_u$ | basic design check for shear |
| $V_u = (q_s)(0.5(b-T)-d)\ell$ | shear load (lb) due to uniform soil-bearing pressure, $q_s$ (psi) |
| $q_s = \dfrac{P_u}{b\ell}$ | uniform solid-bearing pressure (psi) due to factored foundation load $P_u$ (lb) |
| $\phi V_c = \phi 2\sqrt{f'_c}\,\ell d$ | factored shear capacity (lb) |
| $d = t - c - 0.5 d_b$ | depth of reinforcement |
| $\phi = 0.85$ | resistance factor for reinforced concrete in shear |

Two-Way (Punching) Shear

| | |
|---|---|
| $\phi V_c \geq V_u$ | basic design check for shear |
| $V_u = \left(\dfrac{P_u}{b\ell}\right)\left(b\ell - (T+d)^2\right)$ | shear load (lb) due to factored load $P_u$ (lb) |
| $\phi V_c = \phi 4\sqrt{f'_c}\, b_o d$ | factored shear capacity (lb) |
| $b_o = 4(T+d)$ | perimeter of punching shear failure plane around a square column or pier |
| $\phi = 0.85$ | resistance factor for reinforced concrete in shear |

### *Flexure*

The flexure equations that follow pertain specifically to reinforced concrete footings that support walls or square columns. The equations may be generalized for use with other conditions (rectangular footings and rectangular columns, round footings, and so on) by following the same principles. The alternative equation for nominal moment strength $M_n$ is derived from force and moment equilibrium principles by using the provisions of ACI 318. Most designers are familiar with the alternative equation that uses the reinforcement ratio $\rho$ and the nominal strength coefficient of resistance $R_n$. The coefficient is derived from the design check that ensures that the factored moment (due to factored loads) $M_u$ is less than the factored nominal moment strength $\phi M_n$ of the reinforced concrete. To aid the designer in short-cutting these calculations, design manuals provide design tables that correlate the nominal strength coefficient of resistance $R_n$ to the reinforcement ratio $\rho$ for a specific concrete compressive strength and steel yield strength.

ACI 318•15.4

$$\phi M_n \geq M_u$$

basic design check for bending

$$M_u = \frac{1}{8} q_s \ell (b - T)^2$$

factored moment (in-lb) due to soil pressure q(psi) acting on cantilevered portion of the footing s

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

factored nominal moment capacity (in-lb)

$$a = \frac{A_s f_y}{0.85 f'c \ell}$$

(*l* is substituted for the ACI 318 symbol *b* for the concrete beam width and is consistent with the footing dimensioning in figure 4.2)

$$\phi = 0.9$$

resistance factor for reinforced concrete in bending

Alternate method to determine $M_n$

$$\phi M_n = \phi \rho b d f_y \left( d - \frac{0.5 \rho d f_y}{0.85 f'_c} \right)$$

$$\rho = \left( \frac{0.85 f'_c}{f_y} \right) \left( \ell - \sqrt{\frac{2 R_n}{0.85 f'_c}} \right)$$

reinforcement ratio determined by use of $R_n$ nominal strength "coefficient of resistance

$$R_n = \frac{M_u}{\phi \ell d^2}$$

*l* is substituted for the ACI 318 symbol *b* for the concrete beam width and is consistent with the footing dimensioning in figure 4.2)

defines reinforcement ratio ρ

$$A_s = \rho \ell d$$

(*l* is substituted for the ACI 318 symbol *b* for the concrete beam width and is consistent with the footing dimensioning in figure 4.2)

### *Minimum Reinforcement*

Because of concerns with shrinkage and temperature cracking, ACI 318 requires a minimum amount of steel reinforcement. For grade 60 reinforcing steel, the minimum area of steel used for shrinkage and temperature cracking is 0.0018 square inches. ACI 318 requirements on the minimum area of steel for flexural members are shown in the following equations:

ACI 318•7.12, 10.5

$$\rho_{min} = 200/f_y \text{ or } 0.0018 \text{ or } 0.0018$$

$$A_{s,min} = \rho_{min} ld$$

(*l* is substituted for the ACI 318 symbol *b* for the concrete beam width and is consistent with the footing dimensioning in figure 4.2)

Designers often specify one or two longitudinal No. 4 or No. 5 bars for wall footings as nominal reinforcement when building on questionable soils, when required to maintain continuity of stepped footings on sloped sites, or when conditions result in a changed footing depth. For most residential foundations, however, the primary resistance against differential settlement is provided by the deep beam action of the foundation wall, especially if the wall is reinforced masonry or concrete; footing reinforcement may provide limited benefit. In such cases, the footing simply acts as a platform for the wall construction and distributes loads to a larger soil-bearing area.

### *Lap Splices*

Where reinforcement cannot be installed in one length to meet reinforcement requirements, as in continuous wall footings, reinforcement bars must be lapped to develop the bars' full tensile capacity across the splice. In accordance with ACI 318, a minimum lap length of 40 times the diameter of the reinforcement bar is required for splices in the reinforcement. In addition, the separation between spliced or lapped bars must not exceed eight times the diameter of the reinforcement bar or 6 inches, whichever is less. This is a design or construction issue that frequently causes failures during extreme loading conditions from high winds, storm surge, or seismic events. In accordance with TMS 402, the maximum distance between lapped or spliced bars is one-fifth the splice length or 8 inches, whichever is less.

For foundation systems consisting of a plain concrete footing and a plain concrete stem wall, a minimum of one bar should be provided at the top of the stem wall and at the bottom of the footing. Plain concrete footings supporting walls are permitted in Seismic Design categories A, B or C without longitudinal reinforcement. For buildings located in Seismic Categories D or E, the footings should have at least two continuous longitudinal reinforcing bars no smaller than No. 4 and must have a total area of not less than 0.002 times the gross cross-sectional area of the footing. Footings more than 8 inches (203 millimeters) thick must have a minimum of one bar at the top and bottom of the footing. Corners and intersections must have continuity of reinforcement.

# 4.5 Foundation Walls

The objectives of foundation wall design are—

- To transfer the load of the building to the footing or directly to the earth.
- To provide adequate strength, in combination with the footing when required, to prevent differential settlement.
- To provide adequate resistance to shear and bending stresses resulting from lateral soil pressure.
- To provide anchorage for the above-grade structure to resist wind or seismic forces.
- To provide a moisture-resistant barrier to below-ground habitable space, in accordance with the building code.
- To isolate non-moisture-resistant building materials from the ground.

In some cases, masonry or concrete foundation walls incorporate a nominal amount of steel reinforcement to control cracking. Engineering specifications generally require reinforcement of concrete or masonry foundation walls because of somewhat arbitrary limits on minimum steel-to-concrete ratios, even for plain concrete walls. Residential foundation walls are generally constructed of unreinforced or nominally reinforced concrete or masonry or of preservative-treated wood, however. The nominal reinforcement approach has provided many serviceable structures. This section addresses the issue of reinforcement and presents rational design approaches for residential concrete and masonry foundation walls.

In most cases, a designer may select a design for concrete or concrete masonry walls from the prescriptive tables in the applicable residential building code or the IRC (ICC, 2012). Sometimes, however, a specific design applied with reasonable engineering judgment results in a more efficient and economical solution than that prescribed by the codes. The designer may elect to design the wall as either a reinforced or plain concrete wall. The following sections detail design methods for both wall types.

## 4.5.1 Concrete Foundation Walls

Regardless of the type of concrete foundation wall selected, the designer must determine the nominal and factored loads that, in turn, govern the type of wall (that is, reinforced or unreinforced) that may be appropriate for a given application. The following LRFD load combinations suggested for the design of residential concrete foundation walls are based on table 3.1 of chapter 3:

- 1.4 D + 1.6 H
- 1.2 D + 1.6 H + 1.6 L + 0.5 ($L_r$ or S)
- 1.2 D + 1.6 H + 1.6 ($L_r$ or S) + 0.5 L

In light-frame homes, the first load combination typically governs foundation wall design. Axial load increases moment capacity of concrete walls when they are not appreciably eccentric, as is the case in typical residential construction.

To simplify the calculations, the designer may conservatively assume that the foundation wall acts as a simple span beam with pinned ends, although such an assumption will tend to over predict the stresses in the wall. In any event, the simple span model requires that the wall be adequately supported at its top by the connection to the floor framing and at its base by the connection to the footing or bearing against a basement floor slab. Appendix A contains basic load diagrams and beam equations to assist the designer in analyzing typical loading conditions and element-based structural actions encountered in residential design. Once the loads are known, the designer can perform design checks for various stresses by following ACI 318 and the recommendations contained herein.

As a practical consideration, residential designers must keep in mind that concrete foundation walls typically are a nominal 6, 8, or 10 inches thick. The typical concrete compressive strength used in residential construction is 2,500 or 3,000 psi, although other strengths are available. Table 4.4 illustrates recommended minimum concrete compressive strengths based on use and weathering potential. Typical reinforcement tensile yield strength is 60,000 psi (grade 60) and is primarily a matter of market supply (Refer to section 4.2.1 for more information on concrete and steel reinforcement material properties).

***Table 4.4*** ***Minimum Compressive Strength $f'_c$ at 28 Days and Maximum Slump of Concrete***

| **Type or location of concrete construction** | **Weathering Probability** | | | **Maximum slump, in. (mm)** |
|---|---|---|---|---|
| | **Negligible f'c, psi (MPa)** | **Moderate f'c, psi (MPa)** | **Severe f'c, psi (MPa)** | |
| Type 1: Walls and foundations not exposed to weather; interior slabs-on-grade, not including garage floor slabs | 2500 (17) | 2500 (17) | 2,500 (17) | 6(150) |
| Type 2: Walls, foundations, and other concrete work exposed to weather, except as noted below | 2500 (17) | 3000 (21) | 3000 (21) | 6(150) |
| Type 3: Driveways, curbs, walkways, patios, porches, steps, and stairs exposed to weather; garage floors, slabs | 2500 (17) | 3500 (24) | 4500 (31) | 5(125) |

*f'c = minimum compressive strength*
*mm = millimeters.*
*MPa = megapascal*
*psi = pounds per square inch*

### 4.5.1.1 Plain Concrete Wall Design

ACI 318 defines "plain concrete" as structural concrete with no reinforcement or with less reinforcement than the minimum amount specified for reinforced concrete, and ACI 318•22.0 permits its use in wall design. Structural plain concrete basement, foundation, or other walls below the base are permitted in detached one- and two-family, stud-bearing wall dwellings three stories or fewer in height. Plain concrete walls must be used only in regions of low to moderate seismic risk—Seismic Design Category A, B, or C; otherwise, reinforcing is required. ACI 318 recommends incorporating contraction and isolation joints to control cracking; however, this is not a typical practice for residential foundation walls. Temperature and shrinkage cracking is practically unavoidable but is considered to have negligible impact on the structural integrity of a residential wall. Cracking can be controlled (that is, minimizing potential crack widening) by reasonable use of horizontal reinforcement; chapter 4 of the IRC (ICC, 2012) provides some specific prescriptive requirements governing reinforcement size and spacing in plain concrete foundation walls.

ACI 318 limits plain concrete wall thickness to a minimum of 7.5 inches; however, the IRC (ICC, 2012) permits nominal 6-inch-thick foundation walls when the height of unbalanced fill is less than a prescribed maximum.

Adequate strength must be provided and should be demonstrated by analysis, in accordance with the ACI 318 design equations and the recommendations of this section. Depending on soil loads, analysis should confirm conventional residential foundation wall practice in typical conditions (Refer to example 4.3 of section 4.9 for an illustration of a plain concrete foundation wall design).

***Shear Capacity***

Shear stress is a result of the lateral loads on a structure associated with wind, earthquake, or backfill forces. Lateral loads are either normal to the wall surface (that is, perpendicular or out of plane) or parallel to the wall surface (that is, in plane). The designer must consider both perpendicular and parallel shear in the wall.

Perpendicular shear is rarely a controlling factor in the design of residential concrete foundation walls except for some foundation walls with substantial backfill loads. Parallel shear also is usually not a controlling factor in residential foundation walls except for walls that are shear walls resisting lateral loads from high winds or seismic events.

If greater shear capacity is required in a plain concrete wall, increasing the wall thickness or increasing the concrete compressive strength may accomplish that purpose. Alternatively, a wall can be reinforced in accordance with section 4.5.1.2.

The following equations apply to both perpendicular and parallel shear, in conjunction with figure 4.3, for plain concrete walls. For parallel shear, the equations do not address overturning and bending action that occurs in a direction parallel to the wall, particularly for short segments of walls under significant parallel shear load. For concrete foundation walls, that is generally not a concern; for above-grade wood-frame walls, the concern is addressed in chapter 6 in detail.

ACI 318•22.5.4

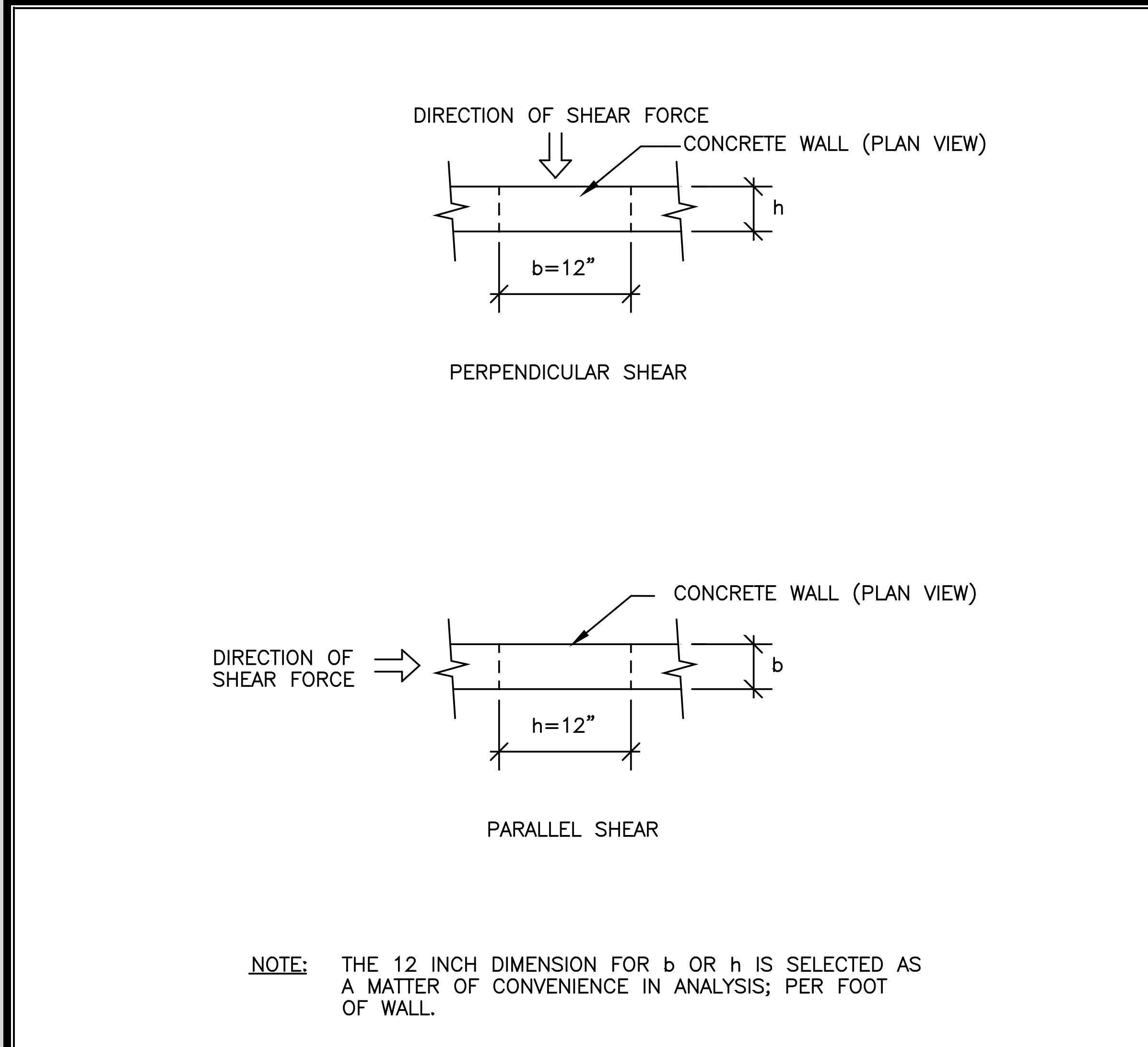
$$V_u \leq \phi V_n$$

$V_u$ = maximum factored shear load on the wall

$$\phi V_n = \phi \frac{4}{3} \sqrt{f'_c}\, bh$$

$$\phi = 0.65$$

***FIGURE 4.3*** ***Shear Calculations for Plain Concrete Walls: Variables Defined***



*Combined Axial Load and Bending Capacity*

The following ACI 318 equations account for the combined effects of axial load and bending moment on a plain concrete wall. The intent is to ensure that the concrete face in compression and the concrete face in tension resulting from factored nominal axial and bending loads do not exceed the factored nominal capacity for concrete. Example 4.4 of section 4.9 demonstrates a method of plotting the interaction equation that follows. (Refer to section 4.5.1.3 for information on interaction diagrams.)

ACI 318•22.5.3, 22.6.3

$$\frac{P_u}{\phi P_n} + \frac{M_u}{\phi M_n} \leq 1 \text{ on the compression face}$$

$$\frac{M_u}{S} - \frac{P_u}{A_g} \leq 5\phi\sqrt{f_c'} \text{ on the tension face}$$

$M_u > M_{u,min}$

$M_u$ = maximum factored nominal moment on wall

$$M_{u,min} = 0.1hP_u$$

$$M_n = 0.85f'_c S$$

$$P_n = 0.6f'_c \left[1 - \left(\frac{l_c}{32h}\right)^2\right] A_g$$

$P_u$ = factored nominal axial load on the wall at point of maximum moment

$$\phi = 0.65$$

Even though a plain concrete wall often calculates as adequate, the designer may elect to add a nominal amount of reinforcement for crack control or other reasons. Walls determined inadequate to withstand combined axial load and bending moment may gain greater capacity through increased wall thickness or increased concrete compressive strength. Alternatively, the wall may be reinforced in accordance with section 4.5.1.2. Walls determined to have adequate strength to withstand shear and combined axial load and bending moment may also be checked for deflection, but this is usually not a limiting factor for typical residential foundation walls.

## 4.5.1.2 Reinforced Concrete Design

ACI 318 allows two approaches to the design of reinforced concrete, with some limits on wall thickness and the minimum amount of steel reinforcement; however, ACI 318 also permits these requirements to be waived in the event that structural analysis demonstrates adequate strength and stability in accordance with ACI 318•14.2.7 (refer to examples 4.4 in section 4.9 for the design of a reinforced concrete foundation wall).

Reinforced concrete walls should be designed in accordance with ACI 318•14.4 by using the strength design method. The following checks for shear and combined flexure and axial load determine if a wall is adequate to resist the applied loads.

### *Shear Capacity*

Shear stress is a result of the lateral loads on a structure associated with wind, earthquake, or lateral soil forces. The loads are either normal to the wall surface (that is, perpendicular or out of plane), however, or parallel to the wall surface (that is, in plane). The designer must check both perpendicular and parallel shear in the wall to determine if the wall can resist the lateral loads present.

If greater shear capacity is required, it may be obtained by (1) increasing the wall thickness, (2) increasing the concrete compressive strength, (3) adding horizontal shear reinforcement, or (4) installing vertical reinforcement to resist shear through shear friction. Shear friction is the transfer of shear through friction between two faces of a crack. Shear friction also relies on resistance from protruding portions of concrete on either side of the crack and by dowel action of the reinforcement that crosses the crack. The maximum limit on reinforcement spacing of 12 or 24 inches specified in ACI 318•11.5.4 is considered to be an arbitrary limit. When reinforcement is required, practical experience dictates 48 inches as an adequate maximum spacing for residential foundation wall design.

The following equations provide checks for both perpendicular and parallel shear in conjunction with figure 4.4. For parallel shear, the equations do not address overturning and bending action that occurs in a direction parallel to the wall, particularly for short segments of walls under significant parallel shear load. For concrete foundation walls, that generally is not a concern; for above-grade wood-framed walls, the topic is addressed in chapter 6 in detail.

ACI 318•11.5,11.7, 11.10

$$V_u \leq \phi V_n$$

$V_u$ = maximum factored shear load on wall

$$V_n = V_c + V_s$$

$$V_c = 2\sqrt{f'_c}\,b_w d$$

$$V_s = \frac{A_v f_y d}{s} \leq 8\sqrt{f'_c}\,b_w d \text{ when } V_u > \phi V_c \text{ when}$$

$$\phi = 0.85$$

Shear-Friction Method

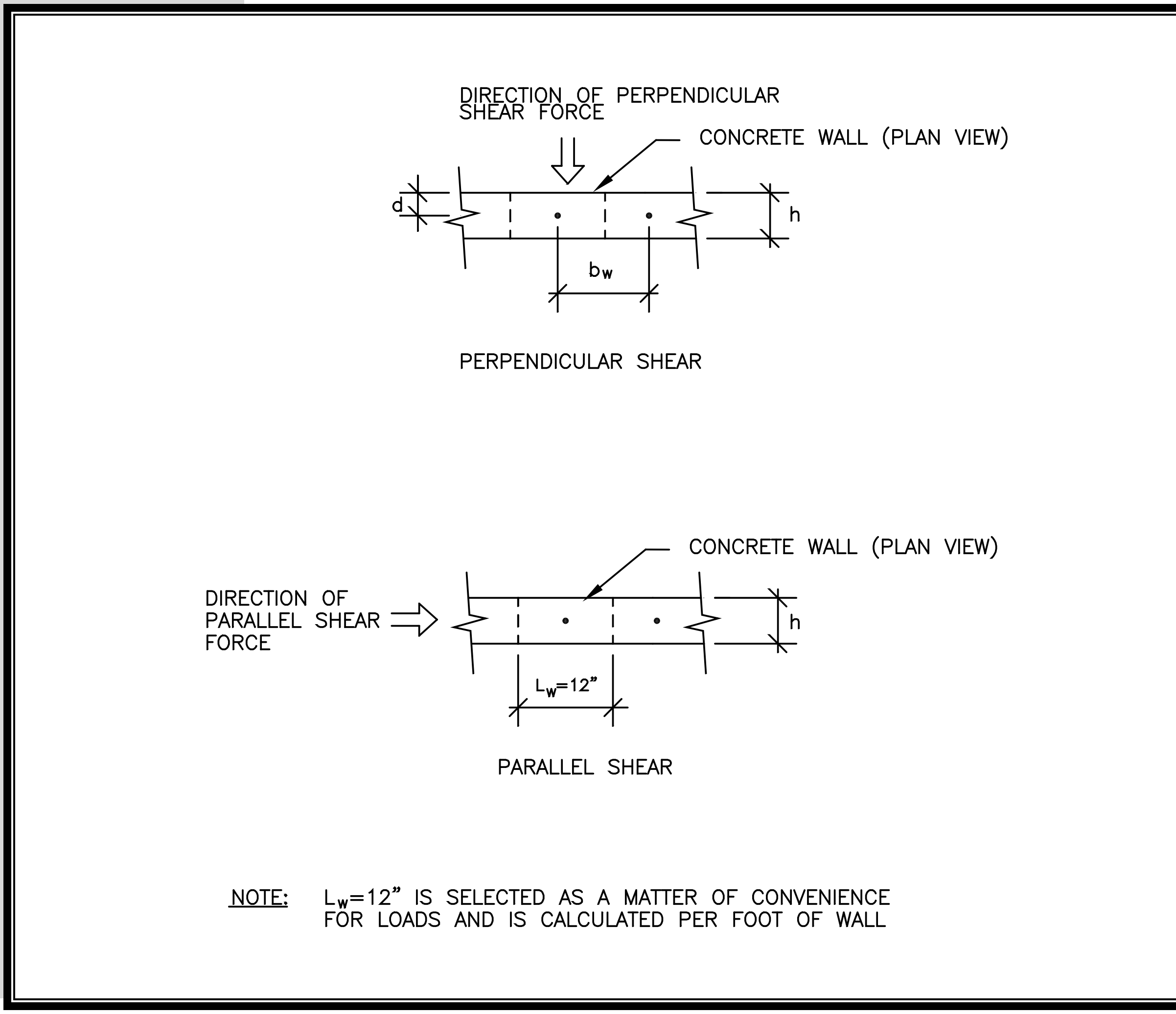
$$V_u \leq \phi V_n$$

$$V_n = A_{vf} f_y \mu \leq 0.2 f'_c A_c \text{ and } \leq 800 A_c \text{ and}$$

$$A_c = b_w h$$

$$\phi = 0.85$$

## FIGURE 4.4 *Shear Calculations in Reinforced Concrete Walls: Variables Defined*



### *Combined Flexural and Axial Load Capacity*

ACI 318 prescribes reinforcement requirements for concrete walls. Foundation walls commonly resist both an applied axial load from the structure above and an applied lateral soil load from backfill. To ensure that the wall's strength is sufficient, the designer must first determine slenderness effects (that is, Euler buckling) in the wall. ACI 318•10.10 provides an approximation method to account for slenderness effects in the wall; however, the slenderness ratio must not be greater than 100. The slenderness ratio is defined in the following section as the ratio between unsupported length and the radius of gyration. In residential construction, the approximation method, more commonly known as the moment magnifier method, is usually adequate because slenderness ratios typically are less than 100 in foundation walls.

The moment magnifier method uses the relationship of the axial load and lateral load in addition to wall thickness and unbraced height to determine a multiplier of 1 or greater, which accounts for slenderness in the wall. The multiplier is termed the *moment magnifier*. It magnifies the calculated moment in the wall resulting from the lateral soil load and any eccentricity in axial load. Together, the axial load and magnified moment are used to determine whether the foundation wall section is adequate to resist the applied loads. The following steps are required to determine the amount of reinforcement required in a typical residential concrete foundation wall to resist combined flexure and axial loads—

- Calculate axial and lateral loads.
- Verify that the nonsway condition applies.
- Calculate slenderness.
- Calculate the moment magnifier.
- Plot the axial load and magnified moment on an interaction diagram.

The following sections discuss the procedure in detail.

### *Slenderness*

Conservatively, assuming that the wall is pinned at the top and bottom, slenderness in the wall can be calculated by using the equation that follows. The effective length factor $k$ is conservatively assumed to equal 1 in this condition. A value of $k$ less than 1 (for example, 0.7) may actually better represent the end conditions (that is, nonpinned state) of residential foundation walls.

ACI 318•10.10

$$\frac{kl_u}{r} < 34 \quad \text{slenderness ratio}$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{bd^3/12}{bd}} = \sqrt{\frac{d^2}{12}} \quad \text{radius of gyration}$$

### *Moment Magnifier Method*

The moment magnifier method is an approximation method allowed in ACI 318•10.10 for concrete walls with a slenderness ratio less than or equal to 100. If the slenderness ratio is less than 34, then the moment magnifier is equal to 1 and requires no additional analysis. The design procedure and equations that follow align with ACI 318•10.12. The equation for *EI*, as listed in ACI 318, is applicable to walls containing a double layer of steel reinforcement. Residential walls typically contain only one layer of steel reinforcement; therefore, the equation for *EI*, as listed herein, is based on section 10.12 (ACI, 2008).

ACI 318•10.12.3

$$M_{u,mag} = \delta M_u \Leftarrow \text{Magnified Moment}$$

$$\delta = \frac{C_m}{1 - \left(P_u / 0.75P_c\right)} \geq 1$$

$$P_c = \frac{\pi^2 EI}{(kl_u)^2}$$

$$C_m = 0.6$$

or

$$C_m = 1 \text{ for members with transverse loads between supports}$$

$$M_{u,min} = P_u(0.6 + 0.03h)$$

$$EI = \frac{0.4E_cI_g}{\beta} \geq \frac{E_cI_g\left(0.5 - e/h\right)}{\beta} \geq \frac{0.1E_cI_g}{\beta}$$

$$e = \frac{M_2}{P_u}$$

$$\beta = 0.9 + 0.5\beta_d^2 - 12\rho \geq 1.0$$

$$\rho = \frac{A_s}{A_g}$$

$$\beta_d = \frac{P_{u,dead}}{P_u}$$

$$E_c = 57{,}000\sqrt{f'_c} \text{ or } w_c^{1.5}33\sqrt{f'_c}$$

Given that the total factored axial load in residential construction typically falls below 3,000 pounds per linear foot of wall and that concrete compressive strength typically is 3,000 psi, table 4.5 provides prescriptive moment magnifiers.

Interpolation is permitted between wall heights and between factored axial loads. Depending on the reinforcement ratio and the eccentricity present, some economy is lost in using the table 4.5 values instead of the preceding calculation method.

***TABLE 4.5*** ***Simplified Moment Magnification Factors, $\delta_{ns}$***

| Minimum Wall Thickness (inches) | Maximum Wall Height (feet) | Factored Axial Load (plf) | |
|---|---|---|---|
| | | 2,000 | 4,000 |
| 5.5 | 8 | 1.07 | 1.15 |
| | 10 | 1.12 | 1.26 |
| 7.5 | 8 | 1.03 | 1.06 |
| | 10 | 1.04 | 1.09 |
| 9.5 | 8 | 1.00 | 1.03 |
| | 10 | 1.00 | 1.04 |

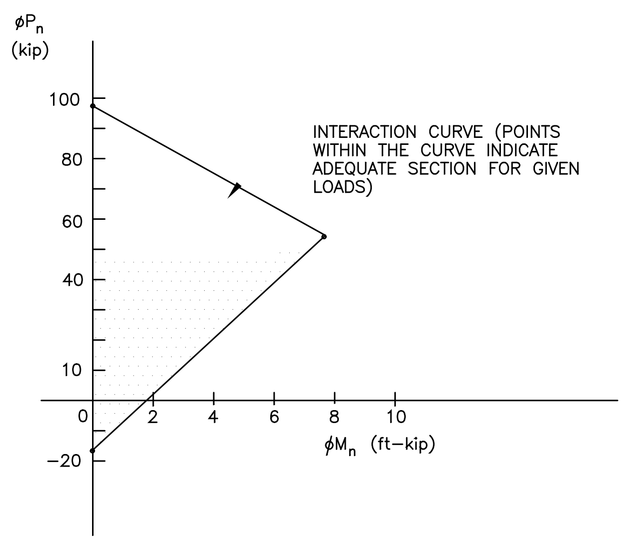
Example 4.7 in section 4.9 presents the complete design of a reinforced concrete foundation wall. The magnified moment and corresponding total factored axial load are plotted on an interaction diagram as shown in figure 4.5 (Refer to section 4.5.1.3 for a description of interaction diagrams and additional resources).

### 4.5.1.3 Interaction Diagrams

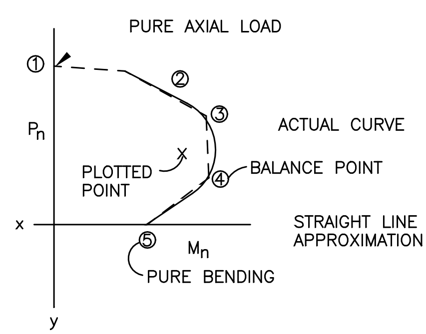
An interaction diagram is a graphic representation of the relationship between the axial load and bending capacity of a reinforced or plain concrete wall. The primary use of interaction diagrams is as a design aid for selecting predetermined concrete wall or column designs for varying loading conditions. Several publications provide interaction diagrams for use with concrete; however, these publications typically focus on column or wall design that is heavily reinforced, in accordance with design loads common in commercial construction. Residential concrete walls are either plain or slightly reinforced with one layer of reinforcement typically placed near the center of the wall. Plain and reinforced concrete interaction diagrams for residential applications and the methods for deriving them may be found in *Structural Design of Insulating Concrete Form Walls in Residential Construction* (PCA, 1998). StructurePoint, an affiliate of the Portland Cement Association (PCA) and the Cement Association of Canada, also offers a computer program that plots interaction diagrams based on user input; the program is titled *spColumn* (PCACOL).

An interaction diagram assists the designer in determining the wall's structural adequacy under various loading conditions (in other words, combinations of axial and bending loads). Figure 4.5 illustrates interaction diagrams for plain and reinforced concrete. Both the design points located within the interaction curve for a given wall height and the reference axes represent a combination of axial load and bending moment that the wall can safely support. The most efficient design is close to the interaction diagram curve. For residential applications, the designer, realizing that the overall design process is not exact, may accept designs within plus or minus 5 percent of the interaction curve.

***FIGURE 4.5*** ***Typical Interaction Diagrams for Plain and Reinforced Concrete Walls***




PLAIN CONCRETE WALL




REINFORCED CONCRETE WALL

Notes:
$\phi P_n$ = factored nominal load
$\phi M_n$ = factored nominal moment
$P_n$ = nominal load
$M_n$ = nominal moment
ft-kip = 1,000 ft-lb

### 4.5.1.4 Minimum Concrete Wall Reinforcement

Plain concrete foundation walls can provide serviceable structures when they are adequately designed (see section 4.5.1.1). When reinforcement is used to provide additional strength in thinner walls or to address more heavily loaded conditions, tests have shown that horizontal and vertical wall reinforcement spacing limited to a maximum of 48 inches on center results in performance that agrees reasonably well with design expectations (Roller, 1996). The designer should still ensure that the reinforcement area meets required minimum specified by the building code and that the reinforcement area is determined by acceptable methods.

ACI 318•22.6.6.5 requires two No. 5 bars around all wall openings. The rebar, at a minimum, should be the same size required by the design of the reinforced wall, or a minimum No. 4 for plain concrete walls. In addition, a lintel (that is, concrete beam) is required at the top of wall openings; refer to section 4.5.1.6 for more detail on lintels.

### 4.5.1.5 Concrete Wall Deflection

ACI 318 does not specifically limit wall deflection; therefore, deflection usually is not analyzed in residential foundation wall design. Regardless, a deflection limit of $L/240$ for unfactored soil loads is not unreasonable for below-grade walls that are reinforced concrete. For plain concrete walls, such large deflections are not tolerable, and designing such walls for strength alone is considered to provide adequate rigidity and serviceability (refer to section 4.4). When using the moment magnifier method, the designer should apply the calculated moment magnification factor to the unfactored load moments used in conducting the deflection calculations. The calculation of wall deflection should also use effective section properties based on $E_cI_g$ for plain concrete walls and $E_cI_e$ for reinforced concrete walls; refer to ACI 318•9.5.2.3 to calculate the effective moment of inertia, $I_e$.

If unfactored load deflections prove unacceptable, the designer may increase the wall thickness or the amount of vertical wall reinforcement. For some residential loading conditions, however, satisfying reasonable deflection requirements should not be a limiting condition.

### 4.5.1.6 Concrete Wall Lintels

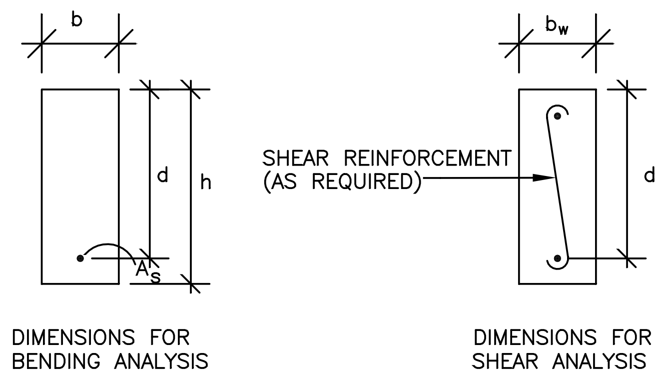
The loads over openings in concrete walls are supported by concrete, steel, precast concrete, cast stone, or reinforced masonry wall lintels. Wood headers also are used when not supporting concrete construction above and when continuity at the top of the wall (that is, a bond beam) is not critical, as in high-hazard seismic or hurricane coastal zones, or is maintained sufficiently by a wood sill plate and other construction above.

This section focuses on the design of concrete lintels in accordance with chapters 10 and 11 of ACI 318. The concrete lintel often is assumed to act as a simple span, with each end pinned; however, the assumption implies no top reinforcement to transfer the moment developed at the end of the lintel. Under that condition, the lintel is assumed to be cracked at the ends, such that the end moment is zero, and the shear must be transferred from the lintel to the wall through the bottom reinforcement.

If the lintel is assumed to act as a fixed-end beam, the top and bottom reinforcement should be sufficiently embedded beyond each side of the opening to fully develop a moment-resisting end in the lintel. Although more complicated to design and construct, a fixed-end beam reduces the maximum bending moment (that is, $wl^2/12$ instead of $wl^2/8$) on the lintel and allows increased spans. A concrete lintel cast in a concrete wall acts somewhere between a true simple span beam and a fixed-end beam. Thus, a designer may design the bottom bar for a simple span condition and the top bar reinforcement for a fixed-end condition (conservative). Often, a No. 4 bar is placed at the top of each wall story to help tie the walls together (serving as a bond beam), which also can serve as the top reinforcement for concrete lintels. Figure 4.6 depicts the cross section and dimensions for analysis of concrete lintels. Example 4.5 demonstrates the design of a concrete lintel; refer to section 4.9.

For additional information on concrete lintels and their design procedure, refer to the *Structural Design of Insulating Concrete Form Walls in Residential Construction* (PCA, 1998) and to *Testing and Design of Lintels Using Insulating Concrete Forms* (HUD, 2000). The latter demonstrates, through testing, that shear reinforcement (that is, stirrups) of concrete lintels is not necessary for short spans (in other words, 3 feet or less) with lintel depths of 8 inches or more. This research also indicates that the minimum reinforcement requirements in ACI 318 for beam design are conservative when a minimum No. 4 rebar is used as bottom reinforcement. Further, lintels with small span-to-depth ratios can be accurately designed as deep beams in accordance with ACI 318 when the minimum reinforcement ratios are met (Refer to ACI 318•11.4).

## *FIGURE 4.6 Design Variables Defined for Lintel Bending and Shear*




### *Flexural Capacity*

The following equations are used to determine the flexural capacity of a reinforced concrete lintel, in conjunction with figure 4.6. An increase in the lintel depth or area of reinforcement is suggested if greater bending capacity is required. As a practical matter, though, lintel thickness is limited to the thickness of the wall in which a lintel is placed. In addition, lintel depth often is limited by the floor-to-floor height and the vertical placement of the opening in the wall. In many cases, therefore, increasing the amount or size of reinforcement is the most practical and economical solution.

ACI 318•10

$$M_u \leq \phi M_n$$

$$M_u = \frac{w\ell^2}{12} \text{ for fixed-end beam model}$$

$$M_u = \frac{w\ell^2}{8} \text{ for simple span beam model}$$

$$\phi M_n = \phi A_s f_y \left( d - \frac{a}{2} \right)$$

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

$$\phi = 0.9$$

***Shear Capacity***

Concrete lintels are designed for shear resulting from wall, roof, and floor loads, in accordance with the equations below and figure 4.6.

ACI 318•11

$$V_u \le \phi V_n$$
$$V_n = V_c + V_s$$
$$V_c = 2\sqrt{f'_c}\, b_w d$$
$$V_s = \frac{A_v f_y d}{s} \le 8\sqrt{f'_c}\, b_w d \text{ when } V_u > \phi V_c$$
$$A_{v,min} = \frac{50 b_w s}{f_y} \text{ when } V_u > \frac{\phi V_c}{2}$$
$$s \le \text{minimum of } \left\{ d/2 \text{ or } 24 \text{ in} \right\}$$
$$s \le \text{minimum of } \left\{ d/4 \text{ or } 12 \text{ in} \right\} \text{ when } V_s > 4\sqrt{f'_c}\, b_w d$$
$$\phi = 0.85$$

***Check Concrete Lintel Deflection***

ACI 318 does not specifically limit lintel deflection; therefore, a reasonable deflection limit of *L/240* for unfactored live loads is suggested. The selection of an appropriate deflection limit, however, is subject to designer discretion. In some applications, a lintel deflection limit of *L/180* with live and dead loads is adequate. A primary consideration is whether the lintel is able to move independently of door and window frames. Calculation of lintel deflection should use unfactored loads and the effective section properties $E_cI_e$ of the assumed concrete section (Refer to ACI 318•9.5.2.3 to calculate the effective moment of inertia $I_e$ of the section).

## 4.5.2 Masonry Foundation Walls

Masonry foundation wall construction is common in residential construction. It is used in a variety of foundation types, including basements, crawl spaces, and slabs-on-grade. For prescriptive design of masonry foundation walls in typical residential applications, a designer or builder may use the IRC (ICC, 2012) or the local residential building code.

ACI 530 develops methods for the design of masonry foundation walls by using allowable stress design; therefore, design loads may be determined according to load combinations presented in chapter 3 as follows:

- D + H
- D + H + 0.75 ($L_r$ or S) + 0.75 L

In light-frame homes, the first load combination typically governs masonry walls for the same reasons stated in section 4.5.1 for concrete foundation walls. To simplify the calculations, the designer may conservatively assume that the wall story acts as a simple span with pinned ends, although such an assumption may tend to overpredict the stresses in the wall (for a discussion on calculating the loads on a structure, refer to chapter 3). Appendix A contains basic load diagrams and equations to assist the designer in calculating typical loading conditions and element-based structural actions encountered in residential design. Further, walls that are determined to have adequate strength to withstand shear and combined axial load and bending moment generally satisfy unspecified deflection requirements; therefore, foundation wall deflection is not discussed in this section. If desired, however, deflection may be considered as discussed in section 4.5.1.5 for concrete foundation walls.

To follow the design procedure, the designer must know the strength properties of various types and grades of masonry, mortar, and grout currently available on the market; section 4.2.2 discusses the material properties. With the loads and material properties known, the designer can then perform design checks for various stresses by following American Concrete Institute's ACI 530 (ACI, 2013). Residential construction rarely involves detailed masonry specifications but rather makes use of standard materials and methods familiar to local suppliers and trades.

An engineer's inspection of a home is hardly ever required or requested under typical residential construction conditions. Inspection should be considered when masonry construction is specified in high-hazard seismic or hurricane-prone areas. ACI 530 makes no distinction between inspected and noninspected masonry walls and, therefore, does not require adjustments in allowable stresses based on level of inspection.

Residential designers should keep in mind that concrete masonry units (that is, block) are readily available in nominal 6-, 8-, 10-, and 12-inch thicknesses. It is generally more economical if the masonry unit compressive strength $f'_m$ ranges between 1,500 and 3,000 psi. The standard block used in residential and light commercial construction is usually rated at 1,900 psi.

### 4.5.2.1 Unreinforced Masonry Design

ACI 530 addresses the design of unreinforced masonry to ensure that unit stresses and flexural stresses in the wall do not exceed certain maximum allowable stresses. ACI 530 provides for two methods of design: an empirical design approach and an ASD approach.

Walls may be designed in accordance with ACI 530•TMS 402 by using the empirical design method under the following conditions:

- The building is not located in Seismic Design Category D or E, as defined in NEHRP 2009 (FEMA, 2009) or ASCE 7-10 (that is, Seismic Zones 3 or 4 in most current and local building codes). (Refer to chapter 3.)

- Foundation walls do not exceed 8 feet in unsupported height.
- The distance between perpendicular vertical or horizontal supports for loadbearing masonry walls is a maximum of 18 times the wall thickness. This limit typically does not apply to residential basements as required in the IRC (ICC, 2012).
- Compressive stresses do not exceed the allowable stresses listed in ACI 530; compressive stresses are determined by dividing the design load by the gross cross-sectional area of the unit, per ACI 530.
- Backfill heights do not exceed those listed in table 4.5.
- Backfill material is nonexpansive and is tamped no more than necessary to prevent excessive settlement.
- Masonry is laid in running bond with Type M or S mortar.
- Lateral support is provided at the top of the foundation wall before backfilling.

Drainage is important when using the empirical table because lack of good drainage may substantially increase the lateral load on the foundation wall if the soil becomes saturated. As required in standard practice, the finish grade around the structure should be adequately sloped (minimum 1 inch of fall per foot of distance from the structure) to drain surface water away from the foundation walls. The backfill material should also be drained to remove ground water from poorly drained soils.

Out-of-plane bracing of the masonry foundation walls can be achieved by providing lateral support from the wood floor framing that is supported by and connected to the wall. The most common method of connection is a wood sill plate anchored to the top of the masonry wall with anchor bolts, and nailing of the floor framing to the sill plate (see chapter 7). Bracing by the floor system should be in place prior to the wall being backfilled.

When the limits of the empirical design method are exceeded, the ASD procedure for unreinforced masonry, as detailed herein, provides a more flexible approach by which walls are designed as compression and bending members, in accordance with ACI 530•2.2.

**TABLE 4.6** ***Nominal Wall Thickness for 8-Foot-High Masonry Foundation Walls[1, 2]***

| Nominal Wall Thickness | Maximum Unbalanced Backfill Height | | |
|---|---|---|---|
| | Hollow Unit Masonry | Solid Unit Masonry | Fully Grouted Unit Masonry |
| 6 inches | 3 | 5 | 5 |
| 8 inches | 5 | 5 | 7 |
| 10 inches | 6 | 7 | 8 |
| 12 inches | 7 | 7 | 8 |

*Source: Modified from the ACI 530• 9.6 by using the IRC (ICC, 2012).*
Notes:
[1]Based on a backfill with an assumed equivalent fluid density of 30 pcf.

[2]Backfill height is measured from the top of the basement slab to the finished exterior grade; wall height is measured from the top of the basement slab to the top of the wall.

The fundamental assumptions, derivation of formulas, and design procedures for ASD are similar to those developed for strength-based design for concrete except that the material properties of masonry are substituted for those of concrete. Allowable masonry stresses used in ASD are expressed in terms of a fraction of the specified compressive strength of the masonry at the age of 28 days: $f'_m$. A typical fraction of the specified compressive strength is 0.25 or 0.33, which equates to a conservative safety factor between 3 and 4 relative to the minimum specified masonry compressive strength. Table 4.7 provides design values for flexural tension stress. As in plain concrete, unreinforced masonry has very low tension capacity. The following design checks are used to determine if an unreinforced masonry wall is structurally adequate (refer to example 4.6 for the design of an unreinforced concrete masonry wall).

**TABLE 4.7** ***Allowable Flexural Tension Stresses ($F_a$) for Allowable Stress Design of Unreinforced Masonry***

| Type of Masonry Unit Construction | Mortar Type M or S | |
|---|---|---|
| | **Portland Cement/Lime (psi)** | **Masonry Cement and Air-Entrained Portland Cement/Lime (psi)** |
| **Normal to Bed Joints** | | |
| Solid | 53 | 32 |
| Hollow[1] | | |
| Ungrouted | 33 | 20 |
| Fully grouted | 86 | 81 |
| **Parallel to Bed Joints in Running Bond** | | |
| Solid | 106 | 64 |
| Hollow | | |
| Ungrouted/partially grouted | 66 | 40 |
| Fully grouted | 106 | 64 |

*Source: Table 2.2.3.2 TMS 402*
Note:
[1]For partially grouted masonry, allowable stresses may be determined on the basis of linear interpolation between fully grouted and ungrouted hollow units, based on the amount of grouting.

### *Shear Capacity*

Shear stress is a result of the lateral loads on the structure associated with wind, earthquakes, or backfill forces. Lateral loads are both normal to the wall surface (that is, perpendicular or out of plane) and parallel to the wall surface (that is, parallel or in plane). Both perpendicular and parallel shear should be checked, as either could be a controlling factor in residential foundation walls.

If greater perpendicular shear capacity is required, it may be obtained by (1) increasing the wall thickness, (2) increasing the masonry unit compressive strength, or (3) adding vertical reinforcement in grouted cells. If greater parallel shear capacity is required, it may be obtained by (1) increasing the wall thickness, (2) reducing the size or numbers of wall openings, or (3) adding horizontal joint reinforcement. Horizontal truss-type joint reinforcement can substantially increase parallel shear capacity, provided that it is installed properly in the horizontal mortar bed joints. If not installed properly, it can create a place of weakness in the wall, particularly in out-of-plane bending of an unreinforced masonry wall.

The equations that follow are used to check perpendicular and parallel shear in masonry walls. The variable $N_v$ is the axial design load acting on the wall at the point of maximum shear. The equations are based on $A_n$, which is the net cross-sectional area of the masonry. For parallel shear, the equations do not address overturning and bending action that occurs in a direction parallel to the wall, particularly for short segments of walls under significant parallel shear load.

ACI 530•2.2.5

$$f_v \le F_v$$

$$f_v = \frac{3V}{2A_n}$$

$$F_v = \text{minimum of} \begin{cases} 1.5\sqrt{f'_m} & \text{for axial and shear members} \\ 120\text{psi} \\ 37\text{ psi} + 0.45\dfrac{N_v}{A_n} & \text{for running bond} \end{cases}$$

### *Axial Compression Capacity*

The following equations from ACI 530•2.3 are used to design masonry walls and columns for compressive loads only. They are based on the net cross-sectional area of the masonry, including grouted and mortared areas.

ACI 530•2.3

Columns

$$P \le P_a$$

$$P_a = \left(0.25f'_m A_n + 0.65A_{st}F_s\right)\left[1-\left(\frac{h}{140r}\right)^2\right] \quad \text{where } h/r \le 99$$

$$P_a = \left(0.25f'_m A_n + 0.65A_{st}F_s\right)\left(\frac{70r}{h}\right)^2 \quad \text{where } h/r > 99$$

$$P_{a,maximum} = F_a A_n$$

$$r = \sqrt{\frac{I}{A_n}}$$

Walls

$$f_a \le F_a$$

$$F_a = \left(0.25f'_m\right)\left[1-\left(\frac{h}{140r}\right)^2\right] \quad \text{where } h/r \le 99$$

ACI 530•2.3

$$f_a = P/A$$

$$F_a = (0.25f'_m)\left(\frac{70r}{h}\right)^2 \text{ where } h/r > 99$$

$$r = \sqrt{\frac{I}{A_n}} \cong \frac{t}{\sqrt{12}}$$

$$P_e = \frac{TT^2 E_m I}{h^2} (1 - 0.577\frac{e}{r})^3$$

$P < 1/4\ P_e$
$E_m = 900\ F'_m$

### *Combined Axial Compression and Flexural Capacity*

The following equations from ACI 530 determine the relationship of the combined effects of axial load and bending moment on a masonry wall.

ACI 530•2.3

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$$

$$f_a = \frac{P}{A_n}$$

$$P \le 0.25P_e$$

$$F_a = (0.25f'_m)\left[1 - \left(\frac{h}{140r}\right)^2\right] \text{for } h/r \le 99$$

$$F_a = (0.25f'_m)\left(\frac{70r}{h}\right)^2 \text{ for } h/r > 99$$

$$r = \sqrt{\frac{I}{A_n}}$$

$$f_b = \frac{M}{S}$$

$$F_b = 0.33f'_m$$

$$P_e = \frac{\pi^2 E_m I}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3$$

$$E_m = 900f'_m$$

ft < Ft
Ft = ACI 530 Table 2.2.3.2

$$f_t = \frac{-P}{A_n} + \frac{M}{S}$$

***Tension Capacity***

ACI 530 provides allowable values for flexural tension transverse to the plane of a masonry wall. Standard principles of engineering mechanics determine the tension stress resulting from the bending moment caused by lateral (that is, soil) loads and offset by axial (that is, dead) loads.

ACI 530•2.3

$$f_t < F_t$$

$$F_t = \text{ACI 530 Table 2.2.3.2}$$

$$f_t = \frac{P}{A_n} + \frac{M}{S}$$

Even though an unreinforced masonry wall may calculate as adequate, the designer may consider adding a nominal amount of reinforcement to control cracking (Refer to section 4.5.2.3 for a discussion on nominal reinforcement).

Walls determined inadequate to withstand combined axial load and bending moment may gain greater capacity through (1) increased wall thickness, (2) increased masonry compressive strength, or (3) the addition of steel reinforcement. Usually the most effective and economical solution for providing greater wall capacity in residential construction is to increase wall thickness, although reinforcement also is common. Section 4.5.2.2 discusses the design procedure for a reinforced masonry wall.

## 4.5.2.2 Reinforced Masonry Design

When unreinforced concrete masonry wall construction does not satisfy all design criteria (load, wall thickness limits, and so on), reinforced walls may be designed by following the ASD procedure or the strength-based design procedure of ACI 530. The ASD procedure outlined herein describes an approach by which walls are designed in accordance with ACI 530•2.3. Although not discussed in detail herein, walls may also be designed by following the strength-based design method specified in ACI 530.

For walls designed in accordance with ACI 530•2.3 using the ASD method, the fundamental assumptions, derivation of formulas, and design procedures are similar to those for design using concrete except that the material properties of masonry are substituted for those of concrete. Allowable masonry stresses used in ASD are expressed in terms of a fraction of the specified compressive strength of the masonry at the age of 28 days, $f'_m$. A typical fraction of the specified compressive strength is 0.25, which equates to a conservative safety factor of 4. The following design checks determine whether a reinforced masonry wall is structurally adequate (refer to example 4.7 for the design of a reinforced concrete masonry wall).

### *Shear Capacity*

Shear stress is a result of lateral loads on the structure associated with wind, earthquakes, or backfill forces. Lateral loads are both normal to the wall surface (that is, perpendicular or out of plane) and parallel to the wall surface (that is, parallel or in plane). Both perpendicular and parallel shear should be checked; perpendicular shear may be a controlling factor in the design of masonry walls, and parallel shear could be a controlling factor if the foundation is partially or fully above grade (such as a walkout basement) with a large number of openings.

The equations that follow check perpendicular and parallel shear in conjunction with figure 4.7. Some building codes include a "j" coefficient in these equations. The "j" coefficient defines the distance between the center of the compression area and the center of the tensile steel area; however, it often is dismissed or approximated as 0.9. If greater parallel shear capacity is required, it may be obtained in a manner similar to that recommended in the previous section for unreinforced masonry design. For parallel shear, the equations do not address overturning and bending action that occurs in a direction parallel to the wall, particularly for short segments of walls under significant parallel shear load.

ACI 530•7.5

$$f_v \leq F_v$$

$$f_v = \frac{V}{A_{nv}}$$

$$F_v = 1.0\sqrt{f'_m} \leq 50 \text{ psi} \quad \text{for flexural members}$$

$$F_v = \frac{1}{3}\left(4 - \frac{M}{Vd}\right)\sqrt{f'_m} \leq \left(80 - 45\frac{M}{Vd}\right)\text{psi}$$

$$\text{for shear walls where } \frac{M}{Vd} < 1$$

$$F_v = 1.0\sqrt{f'_m} \leq 35 \text{ psi}$$

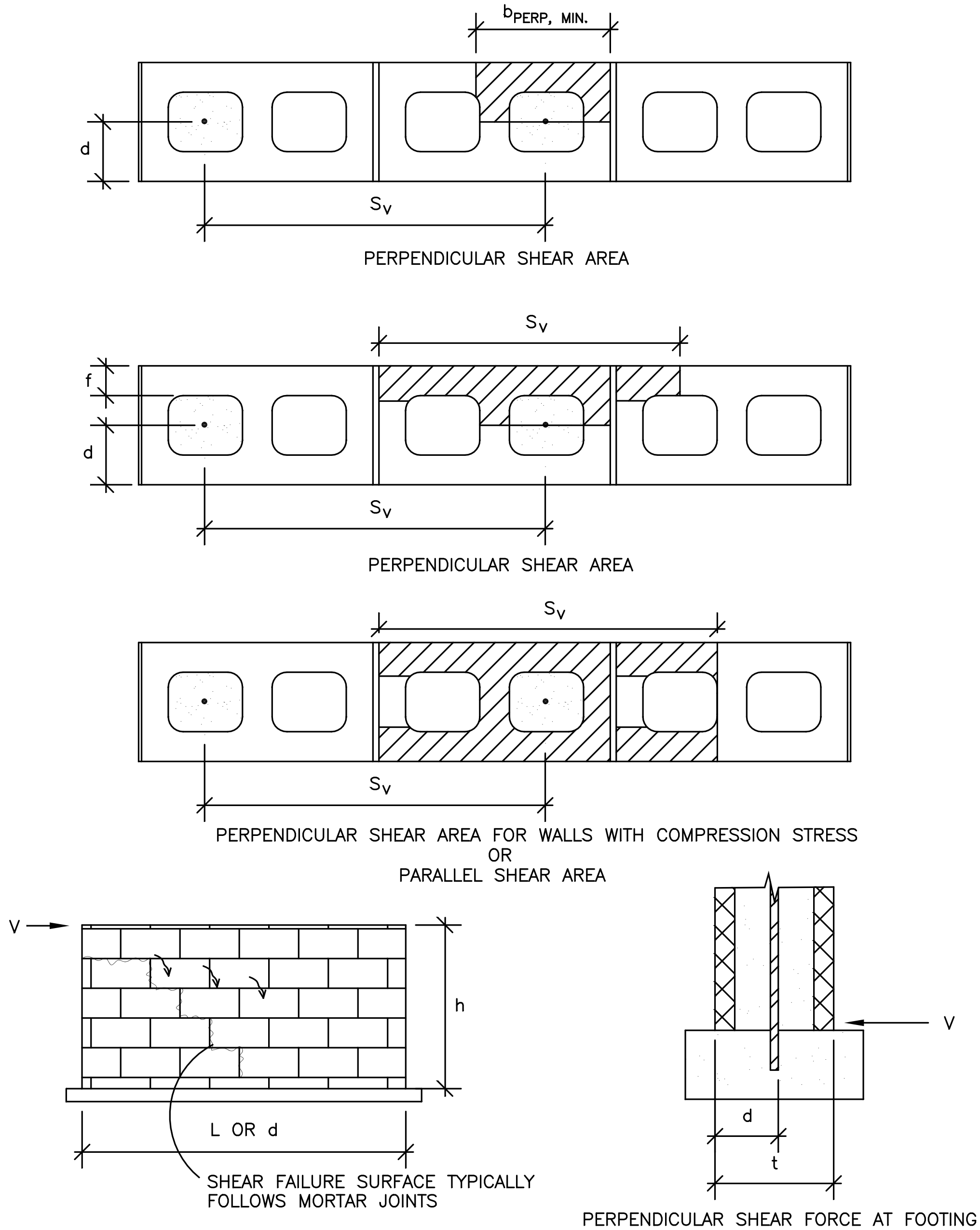
$$\text{for shear walls where } \frac{M}{Vd} \geq 1$$

If the shear stress exceeds the above allowables for masonry only, the designer must design shear reinforcing with the shear stress equation changes, in accordance with ACI 530•2.3.5. In residential construction, increasing the wall thickness or grouting additional cores is generally more economical than using shear reinforcement. If shear reinforcement is desired, refer to ACI 530. ACI 530 limits vertical reinforcement to a maximum spacing (*s*) of 48 inches. Flexural or axial stresses must be accounted for to ensure that a wall is structurally sound. Axial loads increase compressive stresses and reduce tension stresses and may be great enough to keep the masonry in an uncracked state under a simultaneous bending load.

### *Axial Compression Capacity*

The following equations from ACI 530•2.3 are used to determine whether a masonry wall can withstand conditions when compressive loads act only on walls and columns (that is, interior load-bearing wall or floor beam support pier). As with concrete, compressive capacity usually is not an issue in supporting a typical light-frame home. An exception may occur with the bearing points of long-spanning beams. In such a case, the designer should check bearing capacity by using ACI 530•2.1.7.

## FIGURE 4.7 *Shear Calculations in Reinforced Concrete Masonry Walls: Variables Defined*




Notes:
d = distance to neutral axis
L = length
Sv = perpendicular shear area
t = thickness
v = perpendicular shear

ACI 530•2.3

Columns

$$P \le P_a$$

$$P_a = \left(0.25f'_m A_n + 0.65A_{st}F_s\right)\left[1-\left(\frac{h}{140r}\right)^2\right] \quad \text{where } h/r \le 99$$

$$P_a = \left(0.25f'_m A_n + 0.65A_{st}F_s\right)\left(\frac{70r}{h}\right)^2 \quad \text{where } h/r > 99$$

$$P_{a,maximum} = F_a A_n$$

$$r = \sqrt{\frac{I}{A_e}}$$

Walls

$$f_a \le F_a$$

$$F_a = \left(0.25f'_m\right)\left[1-\left(\frac{h}{140r}\right)^2\right] \quad \text{where } h/r \le 99$$

$$F_a = \left(0.25f'_m\right)\left(\frac{70r}{h}\right)^2 \quad \text{where } h/r > 99$$

$$r = \sqrt{\frac{I}{A_e}}$$

Calculation using the preceding equations is based on $A_e$, which is the effective cross-sectional area of the masonry, including grouted and mortared areas substituted for $A_n$.

***Combined Axial Compression and Flexural Capacity***

In accordance with ACI 530•2.3.2, the design tensile forces in the reinforcement due to flexure shall not exceed 20,000 psi for grade 40 or 50 steel, 24,000 psi for grade 60 steel, or 30,000 psi for wire joint reinforcement. As stated, most reinforcing steel in the U.S. market today is grade 60. The following equations pertain to walls that are subject to combined axial and flexure stresses.

ACI 530•7.3

$$F_b = 0.33f'_m$$

$$f_b = \frac{M}{S} \le \left(1-\frac{f_a}{F_a}\right)F_b$$

Columns

$$\frac{P}{P_a} + \frac{f_b}{F_b} \le 1$$

ACI 530•7.3

$$P_a = \left(0.25f'_m A_n + 0.65A_{st}F_s\right)\left[1-\left(\frac{h}{140r}\right)^2\right] \quad \text{where } h/r \le 99$$

$$P_a = \left(0.25f'_m A_n + 0.65A_{st}F_s\right)\left(\frac{70r}{h}\right)^2 \quad \text{where } h/r > 99$$

Walls

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$$

$$f_a = \frac{P}{A_e} \le 0.33f'_m \quad \text{due to flexure only or flexure in combination with axial load}$$

$$F_a = \left(0.25f'_m\right)\left[1-\left(\frac{h}{140r}\right)^2\right] \quad \text{for } h/r \le 99$$

$$F_a = \left(0.25f'_m\right)\left(\frac{70r}{h}\right)^2 \quad \text{for } h/r > 99$$

Walls determined inadequate to withstand combined axial load and bending moment may gain greater capacity through (1) increased wall thickness, (2) increased masonry compressive strength, or (3) added steel reinforcement.

### 4.5.2.3 Minimum Masonry Wall Reinforcement

For reinforced concrete masonry shear walls, ACI 530 stipulates minimum reinforcement limits as shown herein. The designer should rely on experience in local practice and local building code provisions for prescriptive masonry foundation or above-grade wall design in residential applications.

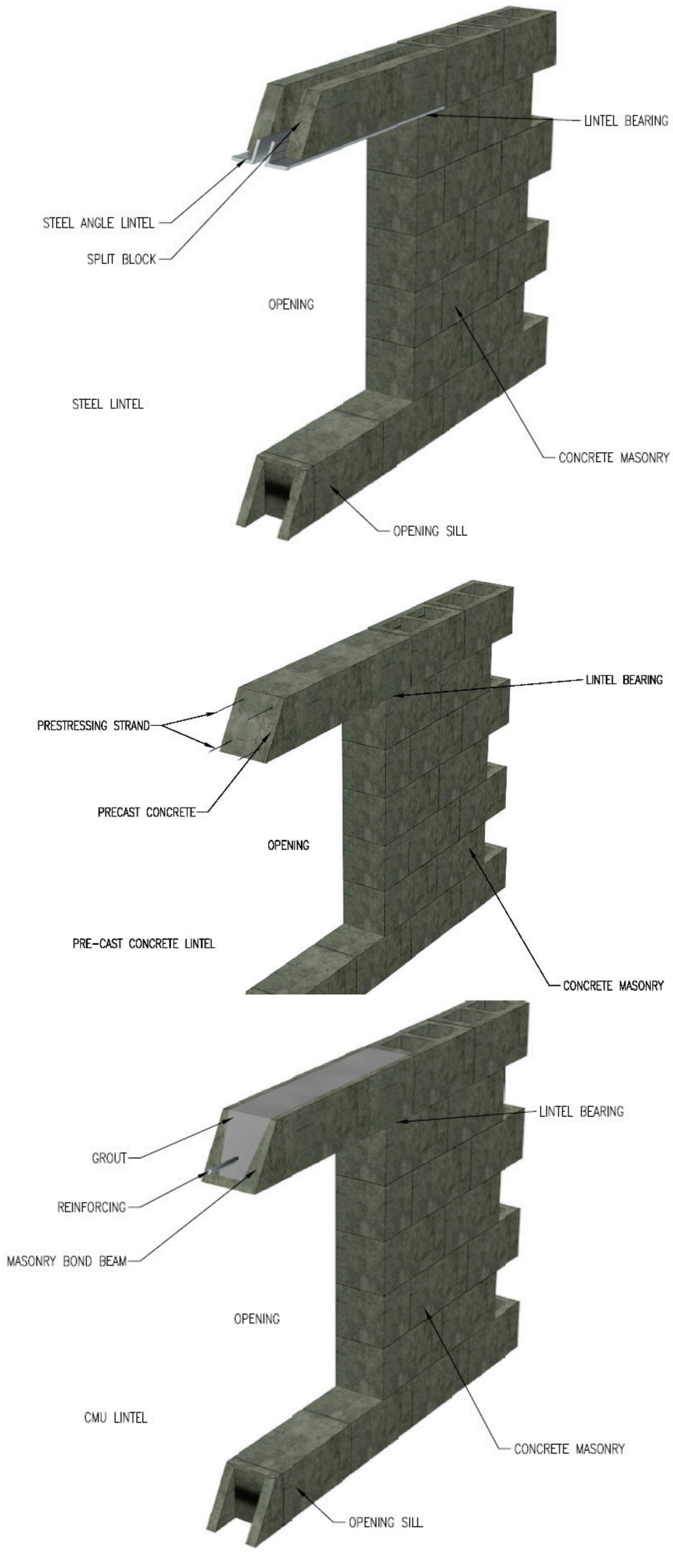
ACI 530•2.3.5

$$A_{s,required} = \frac{M}{F_s d}$$

### 4.5.2.4 Masonry Wall Lintels

Openings in masonry walls are constructed by using steel, precast concrete, or reinforced masonry lintels. Wood headers also are used when they do not support masonry construction above and when continuity at the top of the wall (a bond beam) is not required or is adequately provided within the system of wood-framed construction above. Steel angles are the simplest shapes and are suitable for openings of moderate width typically found in residential foundation walls. The angle should have a horizontal leg of the same width as the thickness of the concrete masonry that it supports. Openings may require vertical reinforcing bars with a hooked end that is placed on each side of the opening to restrain the lintel against uplift forces in high-hazard wind or earthquake regions. Building codes typically require steel lintels exposed to the exterior to be a minimum 1/4-inch thick. Figure 4.8 illustrates some lintels commonly used in residential masonry construction.

**FIGURE 4.8** ***Concrete Masonry Wall Lintel Types***



Many prescriptive design tables are available for lintel design. For more information on lintels, arches, and their design, refer to the National Concrete Masonry Association's (NCMA's) TEK Notes. Information on lintels and arches also can be found in *Masonry Design and Detailing* (Beall, 2012).

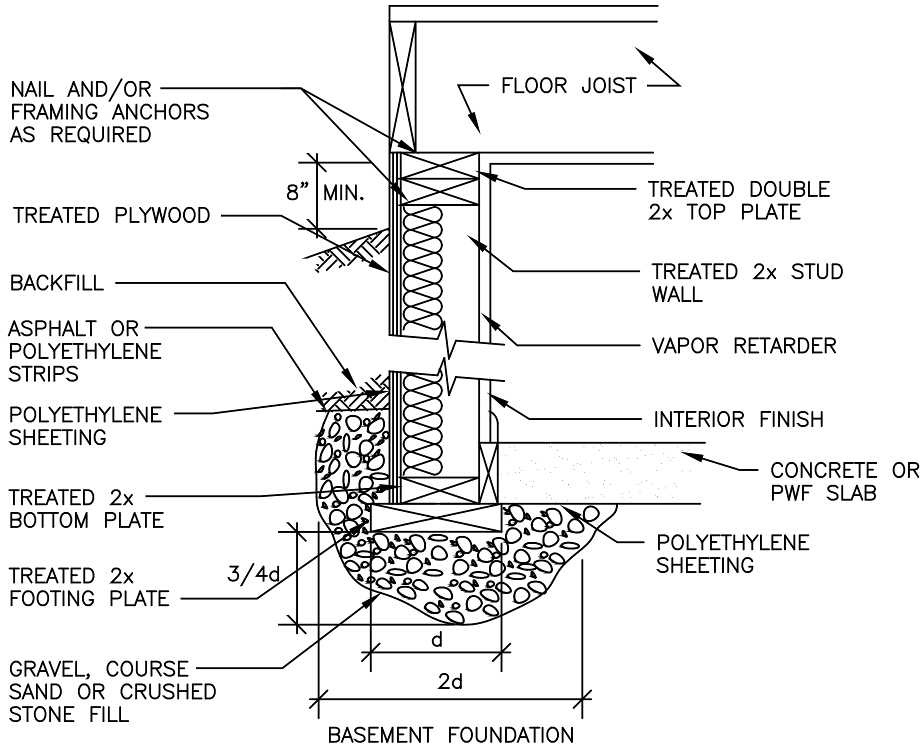
## 4.5.3 Preservative-Treated Wood Foundation Walls

Preservative-treated wood foundations, commonly known as permanent wood foundations (PWF), have been used in more than 300,000 homes and other structures throughout the United States. When properly installed, they provide foundation walls at an affordable cost. In some cases, the manufacturer may offer a 50-year material warranty, which exceeds the warranty offered for other common foundation materials.

A PWF is a load-bearing, preservative-treated, wood-framed foundation wall sheathed with preservative-treated plywood; it bears on a gravel spread footing. PWF lumber and plywood used in foundations are pressure treated with chromated copper arsenate (CCA) or other approved preservatives (AWPA, 2013). The walls are supported laterally at the top by the floor system and at the bottom by a cast-in-place concrete slab, a pressure-treated lumber floor system, or backfill on the inside of the wall. Proper connection details are essential, along with provisions for drainage and moisture protection. All fasteners and hardware used in a PWF should be stainless steel or hot-dipped galvanized steel. Figure 4.9 illustrates a PWF.

PWFs may be designed in accordance with the basic provisions in the IRC (ICC, 2012). Those provisions, in turn, are based on the American Forest and Paper Association's *Permanent Wood Foundation Design Specification* (AF&PA, 2007). The PWF guide offers design flexibility and thorough technical guidance. Table 4.7 summarizes some basic rules of thumb for design, and the steps for using the prescriptive tables follow.

## *FIGURE 4.9 Preservative-Treated Wood Foundation Walls*



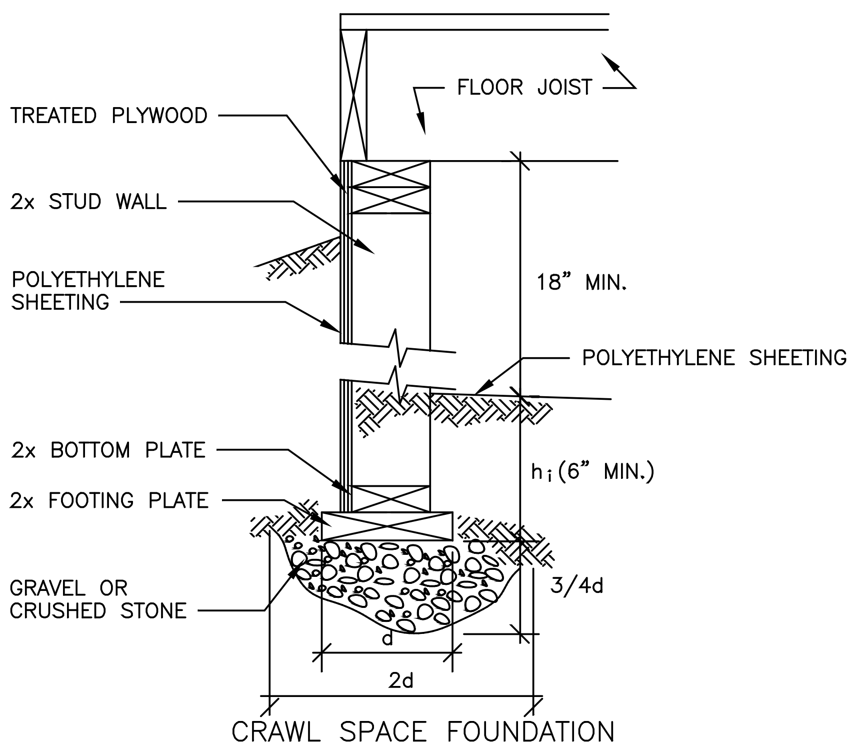



***TABLE 4.8 Preservative-Treated Wood Foundation Framing***[1]

| Maximum Unbalanced Backfill Height (feet) | Nominal Stud Size | Stud Center-to-Center Spacing (inches) |
|---|---|---|
| **5** | 2x6 | 16 |
| **6** | 2x6 | 12 |
| **8** | 2x8 | 12 |

- Connect each stud to top plate with framing anchors when the backfill height is 6 feet or greater.
- Provide full-depth blocking in the outer joist space along the foundation wall when floor joists are oriented parallel to the foundation wall.
- The bottom edge of the foundation studs should bear against a minimum of 2 inches of the perimeter screed board or the basement floor to resist shear forces from the backfill.

[1]Connection of studs to plates and plates to floor framing is critical to the performance of permanent wood foundations. The building code and the *Permanent Wood Foundation Design Specification* (AF&PA, 2007) should be carefully consulted with respect to connections.

- Granular (that is, gravel or crushed rock) footings are sized in accordance with section 4.4.1. Permanent wood foundations may also be placed on poured concrete footings.
- Footing plate size is determined by the vertical load from the structure on the foundation wall and the size of the permanent wood foundation studs.
- The size and spacing of the wall framing is selected from tables for buildings up to 36 feet wide that support one or two stories above grade.
- APA-rated plywood is selected from tables based on unbalanced backfill height and stud spacing. The plywood must be treated with preservatives and rated for below-ground application.
- Drainage systems are selected in accordance with foundation type (for example, basement or crawl space) and soil type. Foundation wall moisture proofing (that is, polyethylene sheeting) also is required.

For more information on preservative-treated wood foundations and their specific design and construction, consult the *Permanent Wood Foundation Specification* (AF&PA, 2007).

## 4.5.4 Insulating Concrete Form Foundation Walls

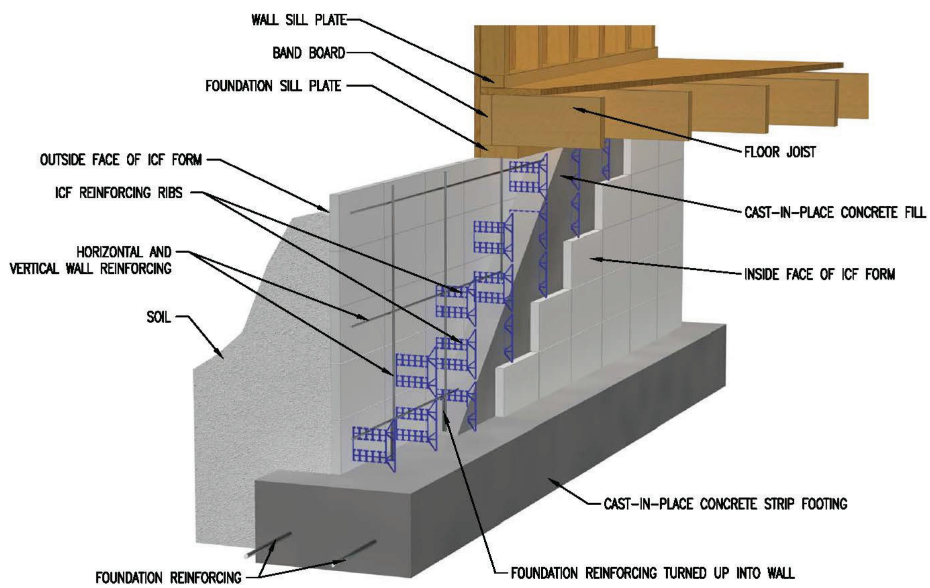
Insulating concrete forms (ICFs) have been used in the United States since the 1970s. They provide durable and thermally efficient foundation and above-grade walls at reasonable cost. ICFs are constructed of rigid foam plastic, composites of cement and plastic foam insulation or wood chips, or other suitable insulating materials that have the ability to act as forms for cast-in-place concrete walls. The forms are easily placed by hand and remain in place after the concrete is cured to provide added insulation.

ICF systems typically are categorized with respect to the form of the ICF unit. There are three types of ICF forms: hollow blocks, planks, and panels. The shape of the concrete wall is best visualized with the form stripped away, exposing the concrete to view. Following are the ICF categories based on the resulting nature of the concrete wall.

- *Flat*. Solid concrete wall of uniform thickness.
- *Post-and-beam*. Concrete frame constructed of vertical and horizontal concrete members with voids between the members created by the form. The spacing of the vertical members may be as great as 8 feet.
- *Screen-grid*. Concrete wall composed of closely spaced vertical and horizontal concrete members with voids between the members created by the form. The wall resembles a thick screen made of concrete.
- *Waffle-grid*. Concrete wall composed of closely spaced vertical and horizontal concrete members with thin concrete webs filling the space between the members. The wall resembles a large waffle made of concrete.

Foundations may be designed in accordance with the values provided in the most recent national building codes' prescriptive tables (ICC, 2012). Manufacturers also usually provide design and construction information. ICF walls are designed by following a procedure similar to that in section 4.5.1; however, special consideration must be given to the dimensions and shape of an ICF wall that is not a flat concrete wall (refer to figure 4.10 for a typical ICF foundation wall detail).

**FIGURE 4.10** ***Insulating Concrete Form Foundation Walls***




For more design information, consult the *Prescriptive Design of Exterior Concrete Walls for One- and Two-Family Dwellings* (PCA-100, 2007) or the *Prescriptive Method for Insulating Concrete Forms in Residential Construction* (HUD, 2002).

# 4.6 Slabs on Grade

The primary objectives of slab-on-grade design are—

- To provide a floor surface with adequate capacity to support all applied loads.
- To provide thickened footings for attachment of the above-grade structure and for transfer of the load to the earth where required.
- To provide a moisture barrier between the earth and the interior of the building.

Many concrete slabs for homes, driveways, garages, and sidewalks are built according to standard thickness recommendations and do not require a specific design unless poor soil conditions, such as expansive clay soils, exist on the site.

For typical loading and soil conditions, floor slabs, driveways, garage floors, and residential sidewalks are built at a nominal 4 inches thick per ACI 302•2.1. Where interior columns and load-bearing walls bear on the slab, the slab typically is thickened and may be nominally reinforced (refer to section 4.4 for footing design procedures). Monolithic slabs may also have thickened edges that provide a footing for structural loads from exterior load-bearing walls. The thickened edges may or may not be reinforced in accordance with the loads and the soil-bearing capacity.

Slab-on-grade foundations often are placed on 2 to 3 inches of washed gravel or sand and a 6-mil (0.006 inch) polyethylene vapor barrier. This recommended practice prevents moisture in the soil from wicking through the slab. The sand or gravel layer acts primarily as a capillary break to soil moisture transport through the soil. If tied into the foundation drain system, the gravel layer also can help provide drainage.

A slab-on-grade greater than 10 feet in any dimension will likely experience cracking from temperature and shrinkage effects that create internal tensile stresses in the concrete. To prevent the cracks from becoming noticeable, the designer usually specifies reinforcement, such as welded wire fabric (WWF) or a fiber-reinforced concrete mix. The location of cracking may be controlled by placing construction joints in the slab at regular intervals or at strategic locations hidden under partitions or under certain floor finishes (that is, carpet).

In poor soils in which reinforcement is required to increase the slab's flexural capacity, the designer should follow conventional reinforced concrete design methods. The Portland Cement Association, Wire Reinforcement Institute (WRI), and U.S. Army Corps of Engineers (USACE) each espouse a different method for the design of plain or reinforced concrete slabs-on-grade.

Presented in chart or tabular format, the PCA method selects a slab thickness in accordance with the applied loads and is based on the concept of one equivalent wheel loading at the center of the slab. PCA design typically does not require structural reinforcement; however, a nominal amount of reinforcement is suggested for minimizing cracks, shrinkage, and temperature effects.

The WRI method selects a slab thickness in accordance with a discrete-element computer model for the slab. The approach graphically accounts for the relative stiffness between grade support and the concrete slab to determine moments in the slab and presents the information in the form of design nomographs.

Presented in charts and tabular format, the USACE method is based on Westergaard's (1926) formulae for edge stresses in a concrete slab. This method assumes that the unloaded portions of the slab help support the slab portions under direct loading.

For further information on the design procedures for each design method mentioned and for unique loading conditions, refer to ACI 360, *Guide to Design of Slabs on Ground* (ACI, 2010), or *Design and Construction of Post-Tensioned Slabs on Ground* (PTI, 2008) for expansive soil conditions.

# 4.7 Pile Foundations

Piles support buildings under a variety of special conditions that make conventional foundation practices impractical or inadvisable. Such conditions include—

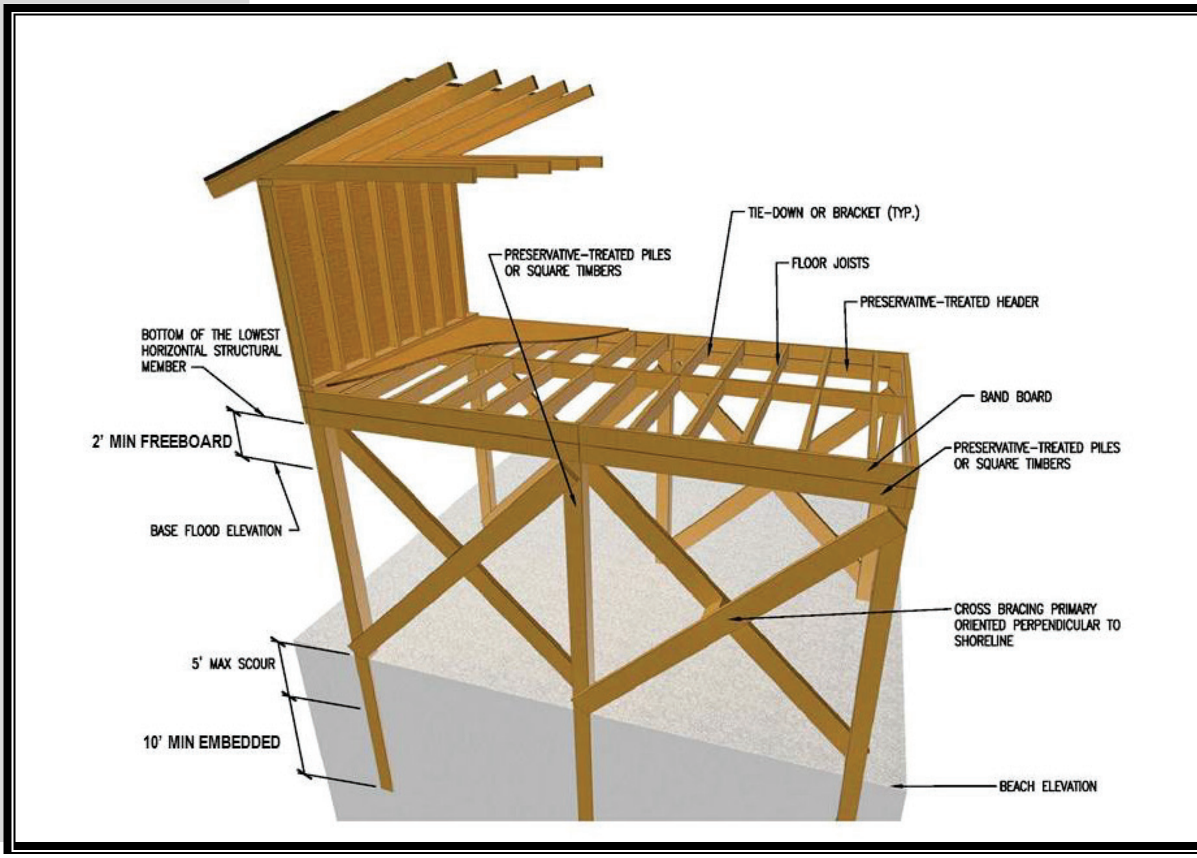
- Weak soils or nonengineered fills that require the use of piles to transfer foundation loads by skin friction or point bearing.
- Inland floodplains and coastal flood hazard zones where buildings must be elevated.
- Steep or unstable slopes.
- Expansive soils on which buildings must be isolated from soil expansion in the "active" surface layer and anchored to stable soil below.

Piles are available in a variety of materials and different mechanisms of support. Preservative-treated timber piles typically are driven into place by a crane with a mechanical or drop hammer (most common in weak soils and coastal construction). Concrete piles or piers typically are cast in place in drilled holes, sometimes with "belled" bases (most common in expansive soils). Steel H-piles or large-diameter pipes are typically driven or vibrated into place with specialized heavy equipment (uncommon in residential construction). Helical piles have screw flights on the end that are "screwed" into the ground until they reach refusal. They most often terminate in a concrete grade beam to tie the tops of the piles together, thereby reducing lateral movement of the foundation system.

Timber piles most commonly are used in light-frame residential construction. The minimum pile capacity is based on the required foundation loading. Pile capacity is, however, difficult to predict; therefore, designers are able to make only rough estimates of required pile lengths and sizes before installation, particularly when the designer relies only on skin friction to develop capacity in deep, soft soils. For this reason, being familiar with local successful practice is a factor in any pile foundation design. A pile foundation sometimes can be specified by drawing on experience, with minimal design effort, in locations not subject to flooding or other extreme loadings from high winds or earthquakes. In other cases, some amount of subsurface exploration (that is, by using a standard penetrometer test) is advisable to assist in foundation design or, alternatively, to indicate when one or more test piles may be required.

Pile depth rarely has to be greater than 8 or 10 feet except in extremely soft soils, on steeply sloped sites with unstable soils, or in coastal hazard areas (that is, beachfront property) where significant scour is possible from storm surge velocity. Under these conditions, depths can easily exceed 15 feet and often reach 25 feet. In coastal high-hazard areas known as "V zones" on flood insurance rating maps (FIRMs), the building must be elevated above the 100-year flood elevation, which is known as the base flood elevation (BFE) and includes an allowance for wave height. Figure 4.11 shows how treated timber piles typically are used to elevate a structure.

**_FIGURE 4.11 Basic Coastal Foundation Construction_**




For additional guidance, the designer should refer to the *Coastal Construction Manual, FEMA P-55* (FEMA, 2011a) and *Home Builder's Guide to Coastal Construction, FEMA P-499* (FEMA, 2011b), both of which are updated frequently by the Federal Emergency Management Agency (FEMA). Another helpful resource is *Pile Driving by Pile Buck* (Pile Buck, 2011). Of course, designers should be prepared to make reasonable design modifications and judgments based on personal experience with and knowledge of pile construction and local conditions. The designer should also carefully consider National Flood Insurance Program (NFIP) requirements because they may affect the availability and cost of insurance. From a life-safety perspective, people often evacuate pile-supported buildings during a major hurricane, but flood damage can be substantial if the building is not properly elevated and detailed. In these conditions, the designer must consider several factors, including flood loads, wind loads, scour, breakaway wall and slab construction, corrosion, and other factors.

The habitable portion of buildings in coastal "A zones" (nonvelocity flow) and inland floodplains must be elevated above the BFE, particularly if owners will be seeking flood insurance. Piles or other forms of an open foundation are the recommended method for constructing a foundation in coastal “A zones.”

The designer must specify a required minimum penetration length and the required axial capacity so the installer can equate driving resistance to sufficient bearing capacity. The designer should use pile capacity formulas such as those provided by the Navy guide titled *Foundations and Earth Structures, Design Manual 7.02* (NAVFAC, 1986). The pile size may be specified as a minimum tip diameter, a minimum butt diameter, or both. The minimum pile butt diameter should be no less than 8 inches; 10- to 12-inch diameters are common. The larger pile diameters may be necessary for unbraced conditions with long, unsupported heights.

In hard material or densely compacted sand or hard clay, a typical pile meets “refusal” when the blows per foot become excessive. In such a case, the builder may need to jet or predrill the pile to a specific depth to meet the minimum embedment and then finish with several hammer blows to ensure that the required capacity is met and the pile properly seated in firm soil. When using either jetting or drilling as an installation method, the designer must consider reducing the capacity of the pile.

Jetting is the process of using a water pump, hose, and long pipe to “jet” the tip of the pile into hard-driving ground, such as firm sand. Jetting may also be used to adjust the pile vertically to maintain a reasonable tolerance with the building layout dimension.

Connecting or anchoring the building properly to pile foundations is important when severe uplift or lateral load conditions are expected. For standard pile and concrete grade beam construction, the pile is usually extended into the concrete “cap” a few inches or more. The connection requirements of the *National Design Specification for Wood Construction* (NDS; AWC, 2012) should be carefully followed for these heavy-duty connections. Such connections are not specifically addressed in chapter 7, although much of the information in that chapter is applicable to the topic.

# 4.8 Frost Protection

The objective of frost protection in foundation design is to prevent damage to the structure from frost action (that is, heaving and thaw weakening) in frost-susceptible soils.

## 4.8.1 Conventional Methods

In northern U.S. climates, builders and designers mitigate the effects of frost heave by constructing homes with perimeter footings that extend below a locally prescribed frost depth. Other construction methods include—

- Piles or caissons extending below the seasonal frost line.

- Mat or reinforced structural slab foundations that resist differential heave.
- Non-frost-susceptible fills and drainage.
- Adjustable foundation supports.

The local building department typically sets required frost depths. Often, the depths set for residential foundations are highly conservative compared with frost depths relevant to other applications. The local design frost depth can vary significantly from that required by actual climate, soil, and application conditions. One exception occurs in Alaska, where it is common to specify different frost depths for "warm," "cold," and "interior" foundations. For homes in the Anchorage, Alaska, area, the perimeter foundation generally is classified as warm, with a required frost depth of 4 to 5 feet. Interior footings may be required to be 8 inches deep. On the other hand, frost depth requirements for cold foundations, including outside columns, may be as much as 10 feet. In the contiguous 48 states, depths for footings range from a minimum 12 inches in the South to as much as 6 feet or more in some northern localities.

Based on the air-freezing index, table 4.8 presents minimum "safe" frost depths for residential foundations. Figure 4.12 depicts the air-freezing index, a climate index closely associated with ground freezing depth. The most frost-susceptible soils are silty soils, or mixtures that contain a large fraction of silt-sized particles. Generally, soils or fill materials with less than 6 percent fines (as measured by a #200 sieve) are considered non-frost-susceptible. Proper surface water and foundation drainage also are important factors where frost heave is a concern. The designer should recognize that many soils may not be frost susceptible in their natural state (such as sand, gravel, or other well-drained soils that are typically low in moisture content). For those soils that are frost susceptible, however, the consequences can be significant and costly if not properly considered in the foundation design.

### *TABLE 4.9 Minimum Frost Depths for Residential Footings*[1, 2]

| Air-Freezing Index (°F-Days) | Footing Depth (inches) |
|---|---|
| 250 or less | 12 |
| 500 | 18 |
| 1,000 | 24 |
| 2,000 | 36 |
| 3,000 | 48 |
| 4,000 | 60 |

[1]Interpolation is permissible.
[2]The values do not apply to mountainous terrain or to Alaska.

## 4.8.2 Frost-Protected Shallow Foundations

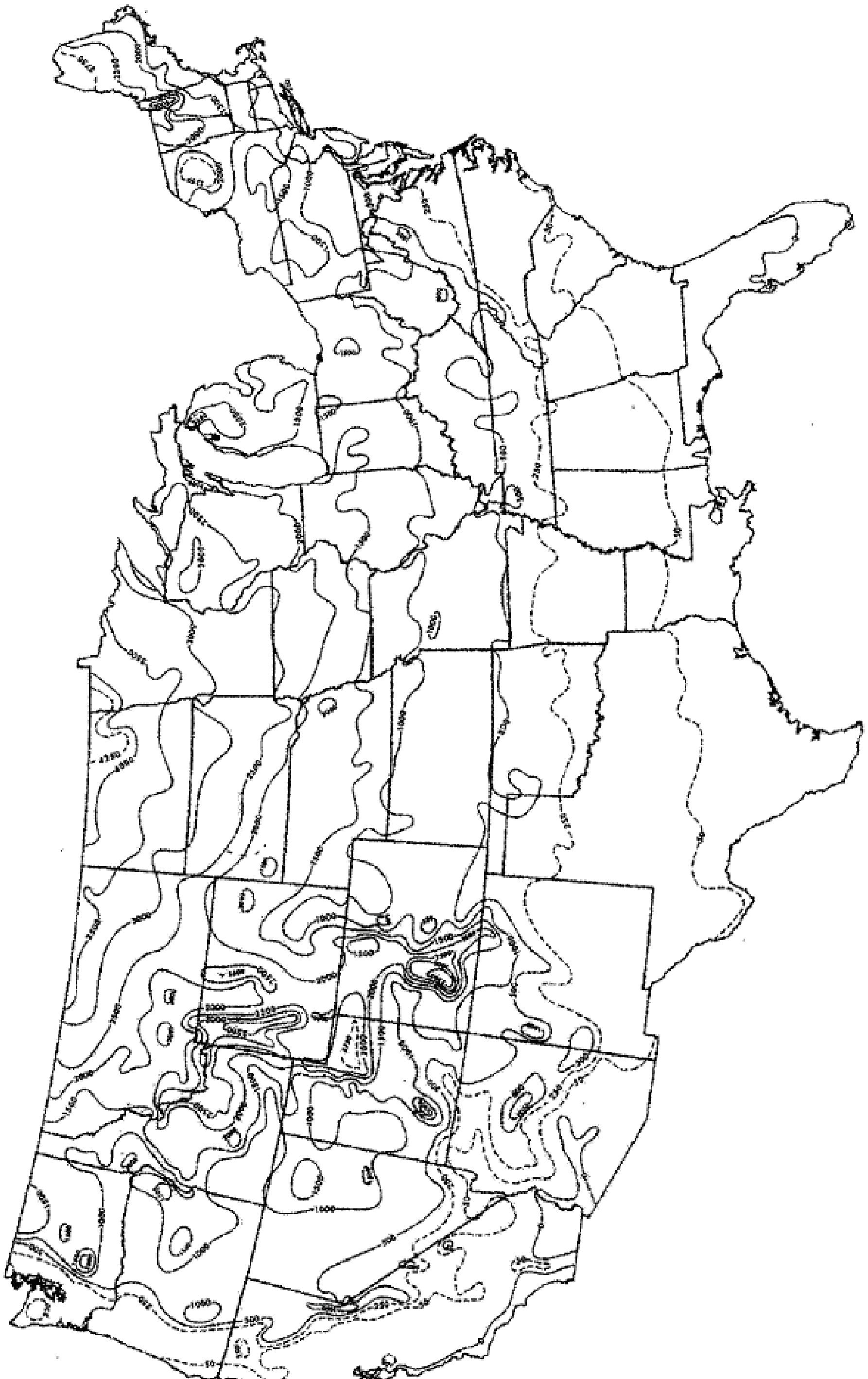
A frost-protected shallow foundation (FPSF) is a practical alternative to deeper foundations in cold regions characterized by seasonal ground freezing and the potential for frost heave. Figure 4.13 illustrates several FPSF applications. FPSFs are best suited to slab-on-grade homes on relatively flat sites. The FPSF method may be used effectively with walkout basements, however, by insulating the foundation on the downhill side of the house, thus eliminating the need for a stepped footing

An FPSF is constructed by using strategically placed vertical and horizontal insulation to insulate the footings around the building, thereby allowing foundation depths as shallow as 12 inches in very cold climates. FPSF technology recognizes earth as a heat source that repels frost. Heat input to the ground from buildings therefore contributes to the thermal environment around the foundation.

The thickness of the insulation and the horizontal distance that the insulation must extend away from the building depends primarily on the climate. In less severe cold climates, horizontal insulation is not necessary. Other factors such as soil thermal conductivity, soil moisture content, and the internal temperature of a building are also important determinants of insulation use. Current design and construction guidelines are based on reasonable worst-case conditions.

After more than 40 years of use in the Scandinavian countries, FPSFs are now recognized in the prescriptive requirements of the IRC (ICC, 2012); however, the code places limits on the use of foam plastic below grade in areas of noticeably high termite infestation probability. In those areas, termite barriers or other modifications must be incorporated into the design to block “hidden” pathways leading from the soil into the structure between the foam insulation and the foundation wall. The exception to the code limit occurs when termite-resistant materials (for example, concrete, steel, or preservative-treated wood) are specified for a home’s structural members.

## FIGURE 4.12 *Air-Freezing Index Map (100-Year Return Period)*



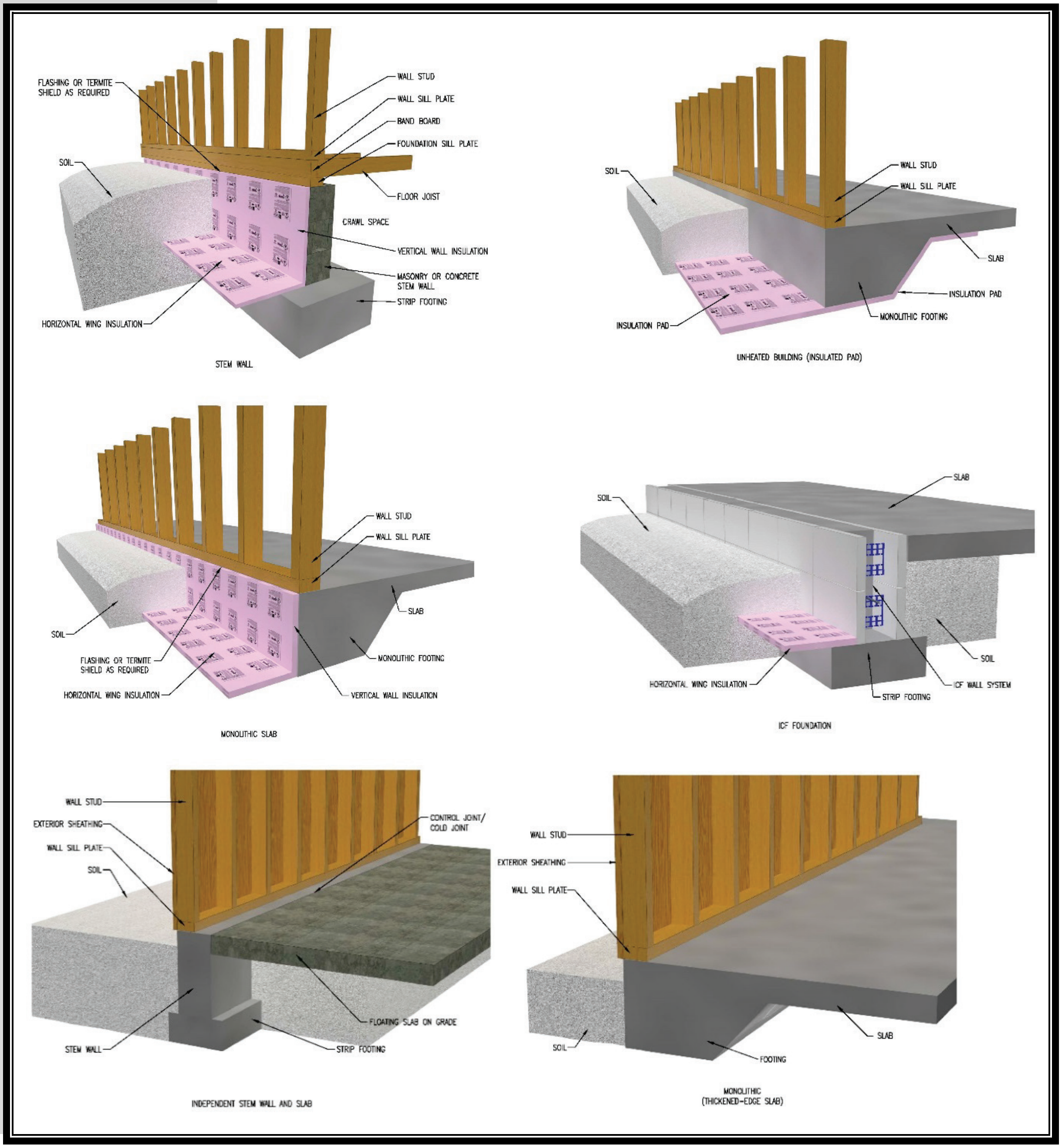
*Source: Reprinted with permission from the International Code Council, Washington, D.C. Copyright ICC.*

*Note: The air-freezing index is defined as the number of cumulative degree days below 32° F and is a measure of the magnitude and duration of below freezing air temperatures.*

The complete design procedure for FPSFs is detailed in *Frost Protected Shallow Foundations in Residential Construction, Second Edition* (NAHB, 1996). The first edition of this guide is available from the U.S. Department of Housing and Urban Development. Either version provides useful construction details and guidelines for determining the amount (thickness) of insulation required for a given climate or application. Acceptable insulation materials include expanded and extruded polystyrenes, although adjusted insulation values are provided for below-ground use.

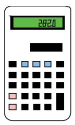
The American Society of Civil Engineers also has a standard for FPSF design and construction based on the resources mentioned. This standard is titled *Design Guide for Frost-Protected Shallow Foundations, ASCE 32-01* (ASCE, 2001).

## *FIGURE 4.13 Frost-Protected Shallow Foundation Applications*



# 4.9 Design Examples

| | |
|---|---|
| ***EXAMPLE 4.1*** | ***Plain Concrete Footing Design*** |



**Given** Exterior continuous wall footing supporting an 8-inch-wide concrete foundation wall carrying a 12-foot floor tributary width; the wall supports two floor levels, each with the same tributary width.

Design Loads

| | | |
|---|---|---|
| Live load | 0.75 [(12 ft)(40 psf) +(12 ft)(30 psf)] = 630 plf | (Table 3.1) |
| Dead load | (12 ft)(10 psf)(2 floors) = 240 plf | (Table 3.2) |
| Wall dead load | (8 ft)(0.66 ft)(150 pcf) = 800 plf | (Table 3.3) |

Footing dead load allowance = 200 plf

Presumptive soil-bearing capacity = 1,500 psf (default)
$f'_c$ = 2,000 psi

**Find** The minimum size of the concrete footing required to support the loads

**Solution**

**1.** Determine the required soil-bearing area.

$$\text{Footing width} = \frac{\text{Design load}}{\text{Soil bearing}} = \frac{(630 + 240 + 800 + 200\ plf)(1\ ft)}{1{,}500\,\text{psf}} = 1.25\,ft$$

The required footing width is equal to

b = 1.25 ft = 15 in ≅ 16 in (standard width of excavation equipment)

**2.** Preliminary design (rule-of-thumb method)

Footing projection = 1/2 (16 in - 8 in) = 4 in

Required plain concrete footing thickness ≅ 4 in (no less than the projection)
∴ Use minimum 6-inch-thick footing.

Footing weight = (1.33 ft)(0.5 ft)(150 pcf) = 100 lb < 200 lb allowance OK

**3.** Consider design options.

- Use 6-inch x 16-inch plain wall concrete footing.
- Design plain concrete footing to check rule of thumb for illustrative purposes only.

Design a plain concrete footing.

(a) Determine soil pressure based on factored loads.

$$q_s = \frac{P_u}{A_{footing}} = \frac{(1.2)(240\text{ plf} + 800\text{plf} + 200\text{plf}) + (1.6)(630\text{plf})}{(1.33\text{ ft})\,(1\text{ ft})} = 1{,}877\text{ psf}$$

(b) Determine thickness of footing based on moment at the face of the wall.

$$M_u = \frac{q_s \ell}{8}(b - T)^2$$

$$= \frac{(1{,}877\text{ psf})(1\text{ ft})}{8}(1.33\text{ ft} - 0.66\text{ ft})^2 = 105\text{ ft} - \text{lb/lf}$$

$$\phi M_n = 5\sqrt{f'_c}\,S = 5\sqrt{2{,}000\text{ psi}}\,\frac{b\ t^2}{6}$$

$$\phi M_n \geq M_u$$

$$(105\text{ ft} - \text{lb/lf})(12\text{ in/ft}) \geq (0.65)(5)\left(\sqrt{2{,}000\text{ psi}}\right)\left(\frac{(12\text{ in})\ t^2}{6}\right)$$

$t = 2.1$ in

(c) Determine footing thickness based on one-way (beam) shear.

$$\phi V_c = \phi \frac{4}{3}\sqrt{f'_c}\,\ell t$$

$$= 0.65\left(\frac{4}{3}\right)\sqrt{2{,}000\text{psi}}\ (12\text{ in})(t)$$

$$V_u = (q_s \ell)(0.5\,(b - T) - t)$$

$$= (1{,}849\text{ psf})(1\text{ ft})(0.5\,(1.33\text{ ft} - 0.66\text{ ft}) - t)$$

$$\phi V_c \geq V_u$$

$$0.65\left(\frac{4}{3}\right)\sqrt{2{,}000\text{ psi}}\ (12\text{ in})(t) = (1{,}877\text{ psf})(1\text{ ft})(0.5(1.33\text{ ft} - 0.66\text{ ft}) - t)$$

$t = 0.27$ ft = 3.2 in

Therefore, shear in the footing governs the footing thickness.

**Conclusion** The calculations yield a footing thickness of 3.2 inches. In accordance with ACI 318•22.4.8, two additional inches must be added, resulting in a footing thickness of 5.2 inches. In accordance with ACI 318•22.7.4, however, plain concrete footings may not have a thickness less than 8 inches. In this case, a more economical and code-compliant footing design (6 inches thick) can be achieved by following the IRC prescriptive provisions for footings rather than following ACI provisions.

In high-hazard seismic areas, a nominal footing reinforcement should be considered (for example, one No. 4 bar longitudinally); however, longitudinal reinforcement at the top and bottom of the foundation wall provides greater strength against differential soil movement in a severe seismic event, particularly on sites with soft soils.

## *EXAMPLE 4.2* — *Reinforced Footing Design*

**Given**

Interior footing supporting a steel pipe column (3.5 in x 3.5 in bearing) carrying a 12-ft x 12-ft floor tributary area

Service Loads

Live load (12 ft)(12 ft)(40 psf) = 5,760 lb
Dead load (12 ft)(12 ft)(10 psf) = 1,440 lb
Footing and column dead load = 300 lb (allowance)

Presumptive soil bearing capacity = 1,500 psf (default)
$f'_c$ = 2,500 psi, $f_y$ = 60,000 psi

**Find**

The minimum size of the concrete footing required to support the loads

**Solution**

**1.** Determine the required soil-bearing area.

$$\text{Area required} = \frac{\text{Service load}}{\text{Presumptive soil bearing}} = \frac{(5{,}760\text{ lb} + 1{,}440\text{ lb} + 300\text{ lb})}{1{,}500\text{ psf}} = 5\text{ ft}^2$$

Assume a square footing

$$b = \sqrt{5\text{ ft}^2} = 2.2\text{ ft} = 26\text{ in}$$

**2.** Preliminary design (rule-of-thumb method)

Footing projection = 1/2 (26 in - 3.5 in) = 11.25 in

∴ Required plain concrete footing thickness ≅ 12 in

Footing weight = (5 ft$^2$)(1 ft)(150 pcf) = 750 lb > 300 lb allowance

∴ Recalculation yields a 28-in x 28-in footing.

**3.** Consider design options.

- Use 12-in x 28-in x 28-in plain concrete footing (5 ft$^3$ of concrete per footing, less expensive).
- Reduce floor column spacing (more but smaller footings, perhaps smaller floor beams, more labor).
- Test soil bearing to see if higher bearing value is feasible (uncertain benefits, but potentially large, perhaps one-half reduction in plain concrete footing size).
- Design a plain concrete footing to determine if a thinner footing is feasible
- Design thinner, reinforced concrete footing (tradeoff with material and labor).

**4.** Design a reinforced concrete footing.

Given Square footing, 28 in x 28 in
$f'_c$= 2,500 psi concrete; 60,000 psi steel

Find Footing thickness and reinforcement

(a) Select trial footing thickness, rebar size, and placement.

$t = 6$ in
$c = 3$ in
$d_b = 0.5$ in (No. 4 rebar)

(b) Calculate the distance from extreme compression fiber to centroid of reinforcement d.

$$d = t - c - 0.5d_b$$
$$= 6 \text{ in} - 3 \text{ in} - 0.5\,(0.5 \text{ in})$$
$$= 2.75 \text{ in}$$

(c) Determine soil pressure based on factored load.

$$q_s = \frac{P_u}{A_{footing}} = \frac{(1.2)(1{,}440\text{ lb} + 300\text{ lb}) + (1.6)(5{,}760\text{ lb})}{5\text{ ft}^2} = 2{,}261\text{ psf}$$

(d) Check one-way (beam) shear in footing for trial footing thickness.

$\phi V_c >> V_u$

$$V_u = \left(\frac{P_u}{b}\right)\left(0.5\left(b - T\right) - d\right) =$$

$$V_u = \left(\frac{11{,}304\text{ lbs}}{28\text{ in}}\right)(0.5\,(28\text{ in} - 3.5\text{ in}) - 2.75\text{ in}) = 3{,}835\text{ lbs}$$

$\phi V_c >> V_u$ OK

(e) Check two-way (punching) shear in trial footing.

$\phi V_c > V_u$

$$= (0.85)(4)\sqrt{2500}\text{ psi}\left(4(3.5\text{ in} + 2.75\text{ in})\right)(2.75\text{ in}) = 11{,}688\text{ lbs}$$

$$V_u = \left(\frac{P_u}{b^2}\right)\left(b^2 - (T + d)^2\right)$$

$$= \frac{11{,}304\text{ lbs}}{(28\text{ in})^2}\left((28\text{ in})^2 - (3.5\text{ in} + 2.75\text{ in})^2\right) = 10{,}741\text{ lbs}$$

OK

(f) Determine reinforcement required for footing, based on critical moment at edge of column.

OK

Use four No. 4 bars where $A_s = 4(0.2\text{ in}^2) = 0.8\text{ in}^2 \geq 0.77\text{ in}^2$ OK

| | | | |
|---|---|---|---|
| | | **Conclusion** | Use minimum 28-in x 28-in x 6-in footing with four No. 4 bars or three No. 5 bars each way in footing. |

| | | | |
|---|---|---|---|
| | | | $f'_c$ = 2,500 psi minimum (concrete)<br>$f_y$ = 60,000 psi minimum (steel reinforcing bar) |

## *EXAMPLE 4.3* *Plain Concrete Foundation Wall Design*

**Given**

Design loads

| | | |
|---|---|---|
| Snow load (S) | = | 280 plf |
| Live load (L) | = | 650 plf |
| Dead load (D) | = | 450 plf |
| Moment at top | = | 0 |
| Concrete weight | = | 150 pcf |
| Backfill material | = | 45 pcf |
| $f'_c$ | = | 3,000 psi |
| Wall thickness | = | 8 in |
| Wall height | = | 8 ft |
| Unbalanced backfill height | = | 7 ft |

Assume axial load is in middle one-third of wall.

**Find**

Verify that an 8-inch-thick plain concrete wall is adequate for the following load combinations from chapter 3 (table 3.1).

- 1.2D + 1.6H
- 1.2D + 1.6H + 1.6L + 0.5 ($L_r$ + S)
- 1.2D + 1.6H = 1.6 ($L_r$ + S) + 0.5L

Only the first load combination will be evaluated because it can be shown to govern the wall design.

**Solution**

**1.** Determine loads.

Equivalent fluid density of backfill soil

Silty clay: w = 100 pcf, $K_a$ = 0.45 (see section 3.5)

$q = K_a w = (0.45)(100\ \text{pcf}) = 45\ \text{pcf}$

Total lateral earth load

$$H = \frac{1}{2} q l^2 = \frac{1}{2}(45\text{pcf})(7\text{ft})^2 = 1{,}103\ \text{plf}$$

$$H = \frac{1}{2} q l^2 = \frac{1}{2}(45\text{pcf})(7\text{ft})^2 = 1{,}103\ \text{plf}$$

$$X_1 = \frac{1}{3} l = \frac{1}{3}\ (7\ \text{ft}) = 2.33\ \text{ft}$$

Maximum shear occurs at bottom of wall (see figure A.1 of appendix A)

$$V_{bottom} = V_1 = \frac{1}{2} q h^2 \left(1 - \frac{h}{3L}\right) = \frac{1}{2}(45\ \text{pcf})(7\ \text{ft})^2 \left(1 - \frac{7\ \text{ft}}{3\,(8\ \text{ft})}\right) = 781\ \text{plf}$$

| | | | |
|---|---|---|---|
| | | | Maximum moment and its location<br>$x = h - \sqrt{h^2 - \frac{2V_1}{q}}$<br>$= 7\text{ ft} - \sqrt{(7\text{ ft})^2 - \frac{2(781\text{ plf})}{45\text{ pcf}}}$<br>= 3.2 ft from base of wall or 4.8 ft from top of wall<br>$M_{max}$ (at x = 3.2 ft) = $V_1x - qhx^2 + qx^3$<br>= (781 plf) (3.2 ft) - (45 pcf)(7 ft)(3.2 ft)$^2$ + (45 pcf)(3.2 ft)$^3$<br>= 1,132 ft-lb/ℓf |
| | | **2.** | Check shear capacity.<br>(a) Factored shear load<br>$V_u$ = 1.6 $V_{bottom}$<br>= 1.6 (781 plf) = 1,250 plf<br>(b) Factored shear resistance<br>$\phi V_n = \phi \frac{4}{3}\sqrt{f'_c}\,bh$<br>$= (0.65)\left(\frac{4}{3}\right)\sqrt{3{,}000\text{ psi}}\,(8\text{ in})(12\text{ in}) = 4{,}557\text{ plf}$<br>(c) Check $\phi V_n \geq V_u$<br>4,557 plf >> 1,250 plf OK |
| | | **3.** | Check combined bending and axial load capacity.<br>(a) Factored loads<br>$M_u$ = 1.6 $M_{max}$ = 1.6 (1,132 ft-lb/lf) = 1,811 ft-lb/lf<br>$P_u$ = 1.2 D<br>$D_{structure}$ = 450 plf (given)<br>$D_{concrete@x} = (150\text{ plf})\left(\frac{8\text{ in}}{12\text{ in / ft}}\right)(8\text{ ft} - 3.23\text{ ft}) = 480\text{ plf}$<br>D = 450 plf + 480 plf = 930 plf<br>$P_u$ = 1.2 (930 plf) = 1,116 plf |
| | | | (b) Determine $M_n$, $M_{min}$, $P_u$<br>$M_n$ = 0.85 $f'_cS$<br>$S = \frac{1}{6}bd^2 = \left(\frac{1}{6}\right)(12\text{ in})(8\text{ in})^2 = 128\text{ in}^3/\text{lf}$<br>$M_n$ = 0.85 (3,000 psi)(128 in$^3$/lf) = 326,400 in-lb/lf = 27,200 ft-lb/lf<br>$M_{min} = 0.1hP_u = 0.1\left(\frac{8\text{ in}}{12\text{ in / lf}}\right)$(1,112 plf) = 74 ft-lb/lf<br>$M_u > M_{min}$ OK |

| | | | |
|---|---|---|---|
| | | | $$P_n = 0.6f'_c\left[1-\left(\frac{L}{32h}\right)^2\right]A_g$$<br>$$= 0.6(3{,}000\text{ psi})\left[1-\left(\frac{(8\text{ ft})\left(\frac{12\text{ in}}{\text{ft}}\right)}{32(8\text{ in})}\right)^2\right](8\text{ in})(12\text{ in}) = 148{,}500\text{ plf}$$ |
| | | | (c) Check combined bending and axial stress equations<br><br>Compression $\frac{P_u}{\Phi P_n} + \frac{M_u}{\Phi M_n} \le 1$<br><br>$$\frac{1{,}116\text{ plf}}{(0.65)(148{,}500\text{ plf})} + \frac{1{,}811\text{ ft}-\text{lb/lf}}{(0.65)(27{,}200\text{ ft}-\text{lb/lf})} \le 1$$<br>$0.11 \le 1$ OK<br><br>$\frac{M_u}{S} - \frac{P_u}{A_g} \le \Phi 5\sqrt{f'_c}$ $\frac{P_u}{\phi P_n} + \frac{M_u}{\phi M_n} \le 1$ $\frac{P_u}{\phi P_n} + \frac{M_u}{\phi M_n} \le 1$<br><br>Tension<br><br>$$\frac{M_u}{S} - \frac{P_u}{A_g} \le \phi 5\sqrt{f'_c}$$<br>$$\frac{1{,}811\text{ ft}-\text{lb/lf}\,(12\text{ in/ft})}{128\text{ in}^3/\text{lf}} - \frac{1{,}116\text{ plf}}{(8\text{ in})(12\text{ in})} \le (0.65)(5)\sqrt{3{,}000\text{ psi}}$$<br>$158 \le 178$ OK<br><br>$\therefore$ No reinforcement required |
| | | **4.** | Check deflection at mid-span (see figure A.1 in appendix A).<br><br>$$= \frac{qL^3}{E_c I_g}\left[\frac{hL}{128} - \frac{L^2}{960} - \frac{h^2}{48} + \frac{h^3}{144L}\right]$$<br><br>$$\frac{(45\text{ pcf})(8\text{ ft})^3}{(3{,}122{,}019\text{psi})\left(\frac{12\text{ in}\,(8\text{ in})^3}{12}\right)}\left[\frac{(7\text{ ft})(8\text{ ft})}{128} - \frac{(8\text{ ft})^2}{960} - \frac{(7\text{ ft})^2}{48} + \frac{(7\text{ ft})^3}{144(8\text{ ft})}\right]\left(\frac{1{,}728\text{in}^3}{\text{ft}^3}\right)$$<br><br>= 0.009 in/lf<br><br>$$\triangle_{all} = \frac{L}{240} = \frac{(8\text{ ft})(12\text{ in/ft})}{240} = 0.4\text{ in/lf}$$<br><br>$\triangle_{max} << \triangle_{all}$ OK |
| | | **Conclusion** | An 8-inch-thick plain concrete wall is adequate under the given conditions.<br><br>The preceding analysis was performed for a given wall thickness. The same equations can be used to solve for the minimum wall thickness that satisfies the requirements for shear, combined bending and axial stress, and deflection. With this approach to the problem, the minimum thickness would be 7.6 inches (controlled by tensile stress under combined bending and axial load). |

| | | | |
|---|---|---|---|
| | | | In the strength-based design approach, the safety margin is related to the use of load and resistance factors. In this problem, the load factor was 1.6 (for a soil load, H) and the resistance factor 0.65 (for tensile bending stress). In terms of a traditional safety factor, an equivalent safety margin is found by 1.6/0.65 = 2.5. It is a fairly conservative safety margin for residential structures and would allow for an equivalent soil fluid density of as much as 113 pcf (45 pcf x 2.5) at the point the concrete tensile capacity based on the minimum concrete compressive strength (as estimated by $5\sqrt{f'_c}$ ) is realized. This capacity would exceed loads that might be expected should the soil become saturated, which would occur under severe flooding on a site that is not well drained.<br><br>The use of reinforcement varies widely as an optional enhancement in residential construction to control cracking and provide some nominal strength benefits. If reinforcement is used as a matter of good practice, one No. 4 bar may be placed as much as 8 feet on center. One horizontal bar may also be placed horizontally at the top of the wall and at mid-height. |

| ***EXAMPLE 4.4*** | ***Reinforced Concrete Foundation Wall Design*** |
|---|---|

**Given** Service loads

| | |
|---|---|
| Live load (L) | = 1000 plf |
| Dead load (D) | = 750 plf |
| Moment at top | = 0 |
| Concrete weight | = 150 pcf |
| Backfill material | = 60 pcf (equivalent fluid density) |
| Wall thickness | = 8 in |
| Wall height | = 10 ft |
| Unbalanced backfill height | = 8 ft |
| $f'_c$ | = 3,000 psi |
| $f_y$ | = 60,000 psi |

Assume axial load is in middle one-third of wall.

**Find** If one No. 5 bar at 24 inches on center vertically is adequate for the load combination, U = 1.2D + 1.6H + 1.6L (chapter 3, table 3.1) when rebar is placed 3 inches from outer face of wall (d=5 in).

**Solution**

**1.** Determine loads.

Total lateral earth load

$$H = \frac{1}{2}ql^2 = \frac{1}{2}(60\text{ pcf})(8\text{ ft})^2 = 1{,}920\text{ plf}$$

$$X = \frac{1}{3}l = \frac{1}{3}(8\text{ ft}) = 2.67\text{ ft}$$

Maximum shear occurs at bottom of wall.

$$\sum M_{top} = 0$$

$$V_{bottom} = \frac{H(L-x)}{L} = \frac{(1{,}920\text{ plf})(10\text{ ft} - 2.67\text{ ft})}{10\text{ ft}} = 1{,}408\text{ plf}$$

Maximum moment and its location

$$X_{max} = \frac{ql - \sqrt{q^2l^2 - 2qV_{bottom}}}{q}$$

$$= \frac{(60\text{ pcf})(8\text{ ft}) - \sqrt{(60\text{ pcf})^2(8\text{ ft})^2 - 2(60\text{ pcf})(1{,}408\text{ plf})}}{60\text{ pcf}}$$

$$=$$

$$X_{max} = 3.87\text{ ft from base of wall or }6.13\text{ ft from top of wall}$$

$$M_{max} = \frac{-qlx_{max}^2}{2} + \frac{qx_{max}^3}{6} + V_{bottom}(x_{max})$$

$$= \frac{-(60\text{ pcf})(8\text{ ft})(3.87\text{ ft})^2}{2} + \frac{(60\text{ pcf})(3.87\text{ ft})^3}{6} + (1{,}408\text{ plf})(3.87\text{ ft})$$

$$= 2{,}434 \text{ ft-lb/lf}$$

**2.** Check shear capacity, assuming no shear reinforcement is required ($V_s$=0).

(a) Factored shear load

$$V_u = 1.6\ V_{bottom}$$
$$= 1.6\ (1{,}408 \text{ plf}) = 2{,}253 \text{ plf}$$

(b) Factored shear resistance

$$\phi V_n = \phi\ (V_c + V_s)$$
$$= \phi\,(2)\sqrt{f'_c}\ b_w d$$
$$= (0.85)\ (2)\ \sqrt{3{,}000\text{ psi}}\ (12\text{ in})\ (5\text{ in}) = 5{,}587\text{ plf}$$

(c) Check $\phi V_n \geq V_u$

5,587 plf >> 2,253 plf OK

**3.** Determine slenderness.

All four foundation walls are concrete with few openings; therefore, the system is a nonsway frame. This is a standard assumption for residential concrete foundation walls.

Slenderness $$r = \sqrt{\frac{I_g}{A_g}} = \sqrt{\frac{\left(\frac{1}{12}\right)(12\text{ in})\ (8\text{ in})^3}{(8\text{ in})(12\text{ in})}} = 2.31$$

$$\frac{kl_u}{r} < 34$$

$$\frac{(1)\,(8\text{ in})\,(12\text{ in})}{2.31} = 41.6 \ \geq\ 34 \ \therefore \text{Use}$$

moment magnifier method

**4.** Determine the magnified moment using the moment magnifier method.

$$P_u = 1.2D + 1.6L = 1.2\ (750\text{ plf}) + 1.6\ (1{,}000\text{ plf}) = 2{,}500\text{ plf}$$

Using the approximated moment magnifiers in table 4.4, the moment magnifier from the table for a 7.5-inch-thick wall, 10 feet high, is between 1.04 and 1.09. For a 9.5-inch-thick wall, the values are between 1 and 1.04.

Through interpolation, $\delta$ = 1.04 for a 2,500 plf axial load.

**5.** Check pure bending.

$$a = \frac{A_s f_y}{0.85 f'_c\ b} = \frac{(0.155\ \text{in}^2)(60{,}000\ \text{psi})}{0.85\ (3{,}000\ \text{psi})(12\ \text{in})} = 0.304$$

$$a =$$

$$\phi M_n = \phi A_s f_y\ (d - \frac{a}{2})$$

$$= 0.9\ (0.155\ \text{in}^2)(60{,}000\ \text{psi})(5\ \text{in} - \frac{0.304\ \text{in}}{2})$$

$$= 40{,}577\ \text{in-lb/lf} = 3{,}381\ \text{ft-lb/lf}$$

$$\phi P_n = 0$$

$$M_u = 2{,}434\ \text{ft-lb/lf from step (1)}$$

$$\delta M_u = 1.04\ (2{,}434\ \text{ft-lb/lf}) = 2{,}531\ \text{ft-lb/lf}$$

By inspection of the interaction diagram, one No. 5 at 24 inches on center is OK because $\delta M_u P_u$ is contained within the interaction curve.

**6.** Check deflection.

$$\triangle_{max} = \left[ -\frac{q(x - L + l)^5}{120} + \frac{ql^3x^3}{36L} + \frac{ql^5x}{120L} - \frac{ql^3Lx}{36} \right] \Big/ E_c I_g$$

$$= \frac{\left(1728\,\text{in}^3\right)}{\text{ft}^3} \left[ \begin{array}{l} -\dfrac{(60\,\text{pcf})(6.13\,\text{ft} - 10\,\text{ft} + 8\,\text{ft})^5}{120} + \dfrac{(60\,\text{pcf})(8\,\text{ft})^3(6.13\,\text{ft})^3}{36(10\,\text{ft})} \\ + \dfrac{(60\,\text{pcf})(8\,\text{ft})^5(6.13\,\text{ft})}{120(10\,\text{ft})} - \dfrac{(60\,\text{pcf})(8\,\text{ft})^3(10\,\text{ft})(6.13\,\text{ft})}{36} \end{array} \right] \Bigg/ (3{,}122{,}019\,\text{psi})\left(\frac{(12\,\text{in})(8\,\text{in})^3}{12}\right)$$

$$= 0.025\ \text{in/lf}$$

$$\triangle_{all} = \frac{L}{240} = \frac{(10\ \text{ft})(12\ \text{in/ft})}{240} = 0.5\ \text{in/lf}$$

$$\triangle_{max} \ll \triangle_{all}\ \ \text{OK}$$

**Conclusion** An 8-inch-thick reinforced concrete wall with one vertical No. 5 bar at 24 inches on center is adequate for the given loading conditions.

This analysis was performed for a given wall thickness and reinforcement spacing. The same equations can be used to solve for the minimum reinforcement that satisfies the requirements for shear, combined bending and axial stress, and deflection. This approach would be suitable for a computer spreadsheet design aid. A packaged computer software program can also be purchased to perform this function; however, certain limitations may prohibit the designer from using design recommendations given in this guide.

The use of horizontal reinforcement varies widely as an optional enhancement. If horizontal reinforcement is used as a matter of preferred practice to control potential cracking, one No. 4 bar placed at the top of the wall and at mid-height typically is sufficient.

## *EXAMPLE 4.5* — *Concrete Lintel*

**Given**

| | | |
|---|---|---|
| $f'_c$ | = | 3,000 psi |
| $f_y$ | = | 60,000 psi |
| Dead load | = | 250 plf |
| Live load | = | 735 plf |
| Span | = | 6.5 ft |
| Lintel width | = | 8 in |
| Lintel depth | = | 12 in |

**Find** Minimum reinforcement required

**Solution**

**1.** Determine reinforcement required for flexure.

$\phi M_n \geq M_u$

$$M_u = \frac{wl^2}{12} = \frac{1.2\ (250\ \text{plf}) + 1.6\ (735\ \text{plf})}{12}(6.5\ \text{ft})^2 = M_u$$

$M_u$ = 5,197 ft-lb = 62,361 in-lb

$\phi M_n = \phi A_s f_y\ (d\text{-}0.5a)$

d = 12-in depth - 1.5-in cover - 0.375-in stirrup = 10.125 in

$$a = \frac{A_s f_y}{0.85 f'_c\ b}$$

set $M_u$ = $\phi M_n$ to solve for $A_s$

$$M_u = \phi A_s f_y \left(d - \frac{1}{2}\left(\frac{A_s f_y}{0.85\, f'_c\ b}\right)\right)$$

$$62{,}364\ \text{in-lb} = (0.9)\ A_s\ (60{,}000\ \text{psi})\left(10.125\ \text{in} - 0.5\left(\frac{A_s\, 60{,}000\ \text{psi}}{0.85(3{,}000\ \text{psi})(12\ \text{in})}\right)\right)$$

$$0 = 546{,}750 A_s - 52{,}941\ A_s^2 - 62{,}364$$

$A_{s,required}$ = 0.115 in$^2$

$\therefore$ Use one No. 4 bar ($A_s$ = 0.20 in$^2$)

Check reinforcement ratio.

$$\rho = \frac{A_s}{bd} = \frac{0.2\ \text{in}^2}{(10.125\ \text{in})(8\ \text{in})} = 0.0025$$

$$\rho_b = \frac{0.85 f'_c \beta_1}{f_y}\left(\frac{87{,}000}{f_y + 87{,}000}\right) = \frac{0.85(3{,}000\ \text{psi})(0.85)}{60{,}000\ \text{psi}}\left(\frac{87{,}000}{60{,}000\ \text{psi} + 87{,}000}\right) = 0.021$$

$\rho_{max} = 0.75\rho_b = 0.75(0.021) = 0.016$

$\rho_{min} = 0.0012$

Because OK

**2.** Determine shear reinforcement.

$\phi V_n \geq V_u$

$V_u = \frac{wl}{2} = \frac{1.2(250\,\text{plf}) + 1.6(735\,\text{plf})}{2} = (6.5\text{ ft}) = 4{,}797\text{ lb}$

= Span-to-depth ratio, = = = 6.5>5 $\therefore$ Regular beam

$\phi V_n = \phi V_c + 0 = \phi 2\sqrt{f'_c}\, b_w d = (0.85)(2)\sqrt{3{,}000\,\text{psi}}(8\,\text{in})(10.125\,\text{in}) = 7{,}542\text{ lb}$

$V_u \leq \frac{\phi V_c}{2} = \frac{7{,}542\,\text{lb}}{2} = 3{,}771\text{ lb} < 4{,}797\,\text{lb}$

$\therefore$ Stirrups are required

Because $\phi V_c > V_u > \frac{\phi V_c}{2}$ only the minimum shear reinforcement must be provided.

$A_{v,min} = \frac{50 b_w s}{f_y} = \frac{(50)(8\,\text{in})\frac{(10.125\text{ in})}{2}}{60{,}000\,\text{psi}} = 0.034\text{ in}^2$

$\therefore$Use No. 3 bars

Shear reinforcement is not needed when $\frac{\phi V_c}{2} > V_u$

3,771 lb = 4,797lb - [1.2(250 plf)+1.6(735 plf)]x

x = 0.70ft

Supply No. 3 shear reinforcement spaced 5 in on center for a distance 0.7 ft from the supports.

**3.** Check deflection.

Find x for transformed area

$h\ x\left(\frac{x}{2}\right) = nA_s(d - x)$

$0.5(8\,\text{in})(x)^2 = \left(\frac{29{,}000{,}000\,\text{psi}}{3{,}122{,}019\,\text{psi}}\right)(0.2\,\text{in}^2)(10.125\,\text{in} - x)$

$0 = 4x^2 + 1.86x - 18.8$

x = 1.95 in

Calculate moment of inertia for cracked section and gross section.

$$I_{CR} = \frac{1}{3}hx^3 + nA_s(d-x)^2$$

$$= \frac{1}{3}(8\ \text{in})(1.95\ \text{in})^3 + (9.29)(0.2\ \text{in}^2)(10.125\ \text{in} - 1.95\ \text{in})^2 = 144\ \text{in}^4$$

$$I_g = \frac{1}{12}bh^3 = \frac{1}{12}(8\,\text{in})(12\,\text{in})^3 = 1{,}152\ \text{in}^4$$

Calculate modulus of rupture

$$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{3{,}000\,\text{psi}} = 411\,\text{psi}$$

Calculate cracking moment

$$M_{cr} = \frac{f_r I_g}{Y_t} = \frac{(411\,\text{psi})(1{,}152\ \text{in}^4)}{(0.5)(12\,\text{in})} = 78{,}912\,\text{in}-\text{lb/lf} = 6.6\,\text{kip}-\text{ft/lf}$$

$$= 10.9\ kNm/m$$

Calculate effective moment of inertia.

Because the cracking moment $M_{cr}$ is larger than the actual moment $M_u$, the section is not cracked; thus, $I_e = I_g$.

Calculate deflection

$$\triangle_{allow} = \frac{l}{240} = \frac{(6.5\,\text{ft})(12\,\text{in/ft})}{240} = 0.33\ \text{in}$$

$$\triangle_{actual} = \frac{5\ wl^4}{384\ E_c I_e}$$

$$\triangle_{i(LL)} = \frac{5(735\ \text{plf})(6.5\ \text{ft})^4}{384(3{,}122{,}019\ \text{psi})(1{,}152\ \text{in}^4)(ft^3/1{,}728\ \text{in}^3)} = 0.008\ \text{in}$$

$$\triangle_{i(DL+20\%LL)} = \frac{5\left(250\,\text{plf} + (0.20)735\,\text{plf} + (150\,\text{pcf})(0.66\,\text{ft})(1\,\text{ft})\right)(6.5\,\text{ft})^4}{384(3{,}122{,}019\,\text{psi})(1{,}152\,\text{in}^4)(\text{ft}^3/1{,}728\,\text{in}^3)} = 0.006\,\text{in}$$

$$\Delta_{LT} = \Delta_{i(LL)} + \lambda\Delta_{i(DL+20\%\,LL)}$$

$$= 0.008\ \text{in} + 2\ (0.0055\ \text{in}) = 0.02\ \text{in}$$

$\triangle_{LT} << \triangle_{allow}$ OK

**Conclusion** The minimum reinforcement bar required for an 8-inch x 12-inch concrete lintel spanning 6.5 feet is one No. 4 bar.

| EXAMPLE 4.6 | Unreinforced Masonry Wall Design |
|---|---|

**Given**

Live load = 1,300 plf
Dead load = 900 plf
Weight of wall = 52.5 psf
Moment at top = 0
Masonry weight = 120 pcf
Backfill material = 30 pcf
$f'_m$ = 1,900 psi
Face shell mortar bedding

Assume axial load is in middle one-third of wall.

**Find**

Verify if a 10-in-thick unreinforced masonry wall is adequate for the ACI 530 load combination and 4 ft of unbalanced fill

$$U = D+H$$

**Solution**

**1.** Determine loads.

Equivalent fluid density of backfill soil (chapter 3)

$$q_s = K_a w = (0.30)(100\text{ pcf}) = 30\text{ pcf}$$

Total lateral earth load

$$R = \tfrac{1}{2} q_s l^2 = \tfrac{1}{2}(30\text{ pcf})(4\text{ ft})^2 = 240\text{ plf}$$

$$x = \tfrac{1}{3}\ell = \tfrac{1}{3} = (4\text{ ft}) = 1.33\text{ ft}$$

Maximum shear occurs at bottom of wall

$$\Sigma M_{top} = 0$$

$$V_{bottom} = \frac{ql^2}{2} - \frac{ql^3}{6L} = \frac{30\text{ pcf }(4\text{ ft})^2}{2} - \frac{30\text{ pcf }(4\text{ ft})^3}{6\,(8\text{ ft})} = 200\text{ plf}$$

Maximum moment and its location

$$x_m = \frac{ql - \sqrt{q^2l^2 - 2qV_{bottom}}}{q}$$

$$x_m = \frac{30\,\text{pcf}\,(4\,\text{ft}) - \sqrt{(30\,\text{pcf})^2(4\,\text{ft})^2 - 2\,(30\,\text{pcf})\,(200\,\text{plf})}}{(30\,\text{pcf})}$$

=2.37 ft from base of wall

$$M_{max} = \frac{qlx_m}{2} + \frac{qx_m^3}{6} + V_{bottom}\ (x_m)$$

$$= -\frac{30\,\text{pcf}\,(4\,\text{ft})(2.37\,\text{ft})^2}{2} + \frac{(30\,\text{pcf})\,(2.37\,\text{ft})^3}{6} + 200\,\text{plf}\ (2.37\,\text{ft})$$

= 204 ft-lb/lf

**2.** Check perpendicular shear.

$$\frac{M}{Vd} = \frac{204\,\text{ft}-\text{lb/lf}\ (12\,\text{in/ft})}{200\,\text{plf}\ (9.625\,\text{in})} = 1.27 > 1$$

$$F_v = \begin{cases} 1.5\sqrt{f'_m} = 1.5\sqrt{1{,}900\,\text{psi}} = 65.4\,\text{psi} \\ 120\,\text{psi} \\ 37\,\text{psi} + 0.45\dfrac{N_v}{A_n} = 37\,\text{psi} + 0.45\dfrac{(900\,\text{plf} + 52.5\,\text{psf}(8\,\text{ft} - 2.37\,\text{ft}))}{33\,\text{in}^2} = 53.3\,\text{psi} \end{cases}$$

$$F_v = 53.3\ \text{psi}$$

$$f_v = \frac{3}{2}\left(\frac{V}{A_n}\right) = 1.5\left(\frac{200\,\text{plf}}{(2\ \text{face shells})(1.375\,\text{in})(12\,\text{in})}\right) = 9.1\,\text{psi}$$

The shear is assumed to be resisted by two face shells because the wall is unreinforced and uncracked.

$f_v < F_v$ OK

**3.** Check axial compression.

$$A_n = \ell(2b) = (12\ \text{in})(2)(1.25\ \text{in}) = 30\ \text{in}^2$$

$$I = \frac{1}{12}bh^3 + Ad^2$$

$$= 2\left[\frac{1}{12}(12\ \text{in})(1.25\ \text{in})^3 + (12\ \text{in})(1.25\ \text{in})\left(\frac{9.625\ \text{in}}{2} - \frac{1.25\ \text{in}}{2}\right)^2\right]$$

$$= 529\ \text{in}^4$$

$$r = \sqrt{I/A_n}\sqrt{\phantom{xxxx}} = 4.00 \text{ in}$$

$$S = \frac{I}{c} = 529 \text{ in}^4 / \tfrac{1}{2}(9.625 \text{ in}) = 110 \text{ in}^3$$

$$\frac{h}{r} = \frac{8 \text{ ft}\,(12\,\frac{\text{in}}{\text{ft}})}{4.00 \text{ in}} = 24 < 99$$

$$F_a = (0.25 f'_m) = (0.25)(1{,}900 \text{ psi}) = \quad 1 - \left[\frac{8\text{ft}\ (12 \text{ in/f})}{140\,(4.00 \text{in})}\right]^2$$

$$= 461 \text{ psi}$$

$$P_{max} = F_a A_n = (461 \text{ psi})(33 \text{ in}^2) = 15{,}214 \text{ plf}$$

$$P = 900 \text{ plf (given for U=D+H)}$$

900 plf < 15,214 plf OK

Check Euler buckling load.

$$E_m = 900f'_m = 900\ (1{,}900 \text{ psi}) = 1.71 \times 10^6 \text{ psi}$$

$$e_k = \frac{S}{A_n} = \frac{118 \text{ in}^3}{33 \text{ in}^2} = 3.57 \text{ in} \quad \text{(kern eccentricity)}$$

$$P_e = \frac{\pi^2 E_m I}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3$$

$$= \frac{\pi^2 (1.71\text{x}10^6 \text{ psi})(567 \text{ in}^4)}{(8 \text{ ft})^2 (12 \text{ in/ft})^2}\left(1 - 0.577\left(\frac{3.57 \text{ in}}{4.14 \text{ in}}\right)\right)^3$$

$$= 131{,}703 \text{ plf}$$

$P \leq P \qquad 0.25P_e$ OK

Euler buckling loads are calculated by using actual eccentricities from gravity loads without including effects of lateral loads.

**4.** Check combined axial compression and flexural capacity.

$M = 204$ ft-lb/lf
$P = 900$ plf

virtual eccentricity $\quad e = = \dfrac{204 \text{ ft} - \text{lb/lf}\,(12\text{in/ft})}{900\text{plf}} = 2.72$ in

kern eccentricity $\quad e_k = \dfrac{S}{A_n} = \dfrac{118\text{in}^3}{33\text{in}^2} = 3.57$ in ← GOVERNS

$e < e_k \quad \therefore$ Assume section is uncracked

$$P_e = \frac{\pi^2 E_m I}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3 \frac{\pi^2 E_m I}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3$$

$$= \frac{\pi^2 (900\,\text{plf})(1{,}900\,\text{psi})(567\,\text{in}^4)}{(8\,\text{ft}\,(12\,\text{in/ft}))^2}\left(1 - 0.577\left(\frac{3.57}{4.14}\right)\right)^3$$

$P_e = 131{,}703$ plf

$P < 0.25$ (131,703 plf) = 32,926 plf OK

$$f_a = \frac{P}{A_n} = \frac{900\ \text{plf}}{33\ \text{in}^2} = 27\ \text{psi}$$

$$f_b = \frac{M}{S} = \frac{(900\ \text{plf})(3.57\ \text{in})\left(\frac{2.37\ \text{ft}}{8\ \text{ft}}\right) + (204\ \text{ft-lb/lf})(12\ \text{in/ft})}{118\ \text{in}^3}$$

$= 29$ psi

$F_a = 462$ psi for $h/r \le 99$

$F_b = 0.33\, f'_m = 0.33$ (1,900 psi) = 627 psi

$$\frac{27\,\text{psi}}{462\,\text{psi}} + \frac{29\,\text{psi}}{627\,\text{psi}} = 0.10 \le 1 \quad \text{OK}$$

**5.** Check tension capacity from table 2.2.3.2 for normal to bed joints, hollow, ungrouted (type M or S mortar).

$F_t \le 25$ psi

$$f_t = -\frac{P}{A_n} + \frac{M}{S} = -\frac{900\ \text{plf}}{33\ \text{in}^2} + \frac{3{,}400\ \text{ft-lb/lf}}{118\ \text{in}^3} = 1.54\ \text{psi}$$

$f_t < F_t$ OK

**6.** Minimum reinforcement.

Horizontal reinforcement at 24 inches on center vertically

**Conclusion** An unreinforced masonry wall is adequate for the ACI 530 load combination evaluated; however, horizontal reinforcement at 16 inches on center may be provided optionally to control potential shrinkage cracking, particularly in long walls (i.e., greater than 20 to 30 feet long).

If openings are present, use lintels and reinforcement as suggested in sections 4.5.2.3 and 4.5.2.4.

Note that the calculations have already been completed and that the maximum backfill height calculated for an 8-inch-thick unreinforced masonry wall using hollow concrete masonry is about 5 feet, with a safety factor of 4.

| EXAMPLE 4.7 | Reinforced Masonry Foundation Wall Design |
|---|---|

**Given**

Live load = 1,300 plf
Dead load = 900 plf
Moment at top = 0
Masonry weight = 120 pcf
Wall weight = 52.5 psf
Backfill material = 45 pcf
$f'_m$ = 2,000 psi
Face shell mortar bedding
Type M or S mortar
Wall is partially grouted, one core is grouted at 24 inches on center
Assume axial load is in middle one-third of wall

**Find**

Verify if one vertical No. 5 bar at 24 inches on center is adequate for a reinforced concrete masonry foundation wall that is 8 feet high with 7 feet of unbalanced backfill for the ACI 530 load combination.

$$U=D+H$$

**Solution**

**1.** Determine loads.

Equivalent fluid density of backfill soil (refer to chapter 3)

$$q = K_aW = (0.45)(100) = 45 \text{ pcf}$$

Total lateral earth load

$$R = \tfrac{1}{2} ql^2 = \tfrac{1}{2}(45 \text{ pcf})(7 \text{ ft})^2 = 1{,}103 \text{ lb}$$

$$X = \tfrac{1}{3}\ell = \tfrac{1}{3}(7 \text{ ft}) = 2.33 \text{ ft}$$

Maximum shear occurs at bottom of wall.

$$\sum M_{top} = 0$$

$$V_{bottom} = \frac{ql^2}{2} - \frac{ql^3}{6L} = \frac{45 \text{ pcf}\,(7 \text{ ft})^2}{2} - \frac{(45 \text{ pcf})\,(7 \text{ ft})^3}{6\,(8 \text{ ft})}$$

$$= 781 \text{ plf}$$

Maximum moment and its location

$$x_m = \frac{ql - \sqrt{q^2l^2 - 2qV_{bottom}}}{q}$$

$$= \frac{(45 \text{ pcf})\,(7 \text{ ft}) - \sqrt{(45 \text{ pcf})^2\,(7 \text{ ft})^2 - 2\,(45 \text{ pcf})\,(781 \text{ plf})}}{45 \text{ pcf}}$$

$$= 3.2 \text{ ft from base of wall}$$

$$M_{max} = \frac{qlx_m^{\;2}}{2} + \frac{qx_m^{\;3}}{6} + V_{bottom}(x_m)$$

$$= \frac{-45\,\text{pcf}\,(7\,\text{ft})(3.2\,\text{ft})^2}{2} + \frac{(45\,\text{pcf})\,(3.2\,\text{ft})^3}{6} + (781\,\text{plf})(3.2\,\text{ft})$$

$$= 1{,}132 \text{ ft-lb/lf}$$

**2.** Check perpendicular shear.

$$\frac{M}{Vd} = \frac{1{,}132\,\text{ft}-\text{lb}/\text{lf}\,(12\,\text{in}/\text{ft})}{(781\,\text{plf})(9.625\,\text{in})} = 1.8 > 1$$

$$F_v = 1\sqrt{f'_m} \quad \leq \quad 50 \text{ psi}$$

$$= 1\sqrt{2{,}000\text{psi}} \quad = 44.7\text{psi} < 50 \text{ psi}$$

$$F_v = (44.7 \text{ psi})(2\text{-ft grouted core spacing}) = 89 \text{ psi}$$

$$A_e = A_{\text{CMU faceshells}} + A_{\text{core}}$$

$$= (24 \text{ in} - 8.375 \text{ in})(2)(1.375 \text{ in}) + (1.125 \text{ in}+1.375 \text{ in}+5.875 \text{ in})(9.625 \text{ in})$$

$$= 124 \text{ in}^2$$

$$f_v = \frac{V}{bd} = \frac{V}{A_e} = \frac{(781\,\text{plf})\,(2\text{ft rebar spacing})}{(124\,\text{in}^2)} = 13 \text{ psi}$$

$$f_v < F_v \text{ OK}$$

This assumes that both mortared face shells are in compression.

**3.** Check parallel shear.

Foundation walls are constrained against lateral loads by the passive pressure of the soil and soil-wall friction. Parallel shear on the foundation wall can be neglected by design inspection.

**4.** Check axial compression.

$$A_e = 124 \text{ in}^2$$

$$I = \frac{1}{12}bh^3 + Ad^2$$

$$= \frac{1}{12} \quad = \quad (8.375 \text{ in})(9.625 \text{ in} - 2(1.375 \text{ in}))$$

$$+ 2\left[\left(\frac{1}{12}\right)(24 \text{ in})(1.375 \text{ in})^3 + (24 \text{ in})(1.375 \text{ in})\left(\frac{9.625 \text{ in}}{2} - \frac{1.375 \text{ in}}{2}\right)^2\right]$$

$$= 1{,}138 \text{ in}^4$$

$$r = \sqrt{\frac{I}{A_e}} = = 3.03 \text{ in}$$

$$\frac{h}{r} = \frac{8\,\text{ft}\,(12\,\text{in}/\text{ft})}{3.03\,\text{in}} = 32 < 99$$

$$\therefore F_a = (0.25\ f'_m)\left(1 - \left(\frac{h}{140r}\right)^2\right)$$

$$= 0.25\ (2{,}000\ \text{psi})\left(1 - \left(\frac{(8\,\text{ft})(12\,\text{in}/\text{ft})}{140(3.03\,\text{in})}\right)^2\right) = 474\ \text{psi}$$

$P_{max} = F_a A_e$ = (474 psi)(124 in$^2$)=58,776 lb

P = 900 lb

P < $P_{max}$ OK

**5.** Check combined axial compression and flexural capacity.

M = 1,132 ft-lb/lf

P = 900 plf

virtual eccentricity = e = $M/P$ $M/P$

$$= \frac{1{,}132\ \text{ft} - \text{lb}/\text{lf}\,(12\,\text{in}/\text{ft})}{900\text{plf}} = 15\ \text{in} \leftarrow \text{Governs}$$

kern eccentricity = $e_k$ = $\frac{S}{A_e}$ $\frac{S}{A_e}$

$$= \frac{1{,}138\,\text{in}^4 / 0.5(9.625\,\text{in})}{124\,\text{in}^2} = 1.9\ \text{in}$$

e > $e_k$ $\therefore$ Tension on section, assume cracked

$$f_a = P/A_e = \frac{900\,\text{plf}\,(2\text{ft})}{124\,\text{in}^2} = 14.5\ \text{psi}$$

$$f_b = \frac{M}{S} = \frac{1{,}132\ \text{ft} - \text{lb}/\text{lf}(12\,\text{in}/\text{ft})}{236.5\,\text{in}^3} = 57\ \text{psi}$$

$f_b$ > $f_a$

$\therefore$ Assume section is cracked

$$F_a = 0.25\ f'_m\left[1 - \left(\frac{h}{140r)}\right)^2\right]\left[1 - \left(\frac{h}{140r)}\right)^2\right]$$

$$= 0.25\ (2{,}000\ \text{psi}) \left[ 1 - \left( {}^{8\ \text{ft}(12\ \text{in}/\text{ft})} \Big/ {}_{140\,(3.03\ \text{in})} \right)^2 \right]$$

$= 474$ psi

$F_b = 0.33\ f'_m = 0.33\ (2{,}000\ \text{psi}) = 660\ \text{psi}$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{14.5\,\text{psi}}{474\,\text{psi}} + = 0.12 \leq 1 \text{ OK}$$

**6.** Check tension.

$$\begin{aligned} M_t &= A_s d F_s \\ &= (0.155\ \text{in}^2)(0.5)(9.625\ \text{in})(24{,}000\ \text{psi}) \\ &= 17{,}903\ \text{in-lb/lf} \\ M &= (1{,}132\ \text{ft-lb/lf})(12\ \text{in/ft}) \\ &= 13{,}584\ \text{in-lb/lf} \end{aligned}$$

$M < M_t$ OK

**Conclusion** One vertical No. 5 bar at 24 inches on center is adequate for the given loading combination. In addition, horizontal truss-type reinforcement is recommended at 24 inches (that is, every third course of block).

Load combination D+H controls design; therefore, a check of D+L+H is not shown.

Table 4.5 would allow a 10-inch-thick solid unit masonry wall without rebar in soil with 30 pcf equivalent fluid density. This practice has succeeded in residential construction except as reported in places with “heavy” clay soils; therefore, a design as shown in this example may be replaced by a design in accordance with the applicable residential codes’ prescriptive requirements. The reasons for the apparent inconsistency may be attributed to a conservative soil pressure assumption or a conservative safety factor in ACI 530 relative to typical residential conditions.

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